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CONTENTS

PAPERS

No.		PAGE
1697	**NEW THEORY FOR THE CENTRIFUGAL PUMP.**	
	By **Allen F. Sherzer**	1
	Discussion:	
	By J. W. MacMeeken	30
	Thomson Kingsford	33
	Sanford A. Moss	34
	P. L. Reed	35
	Max M. Munk	37
	Elmo G. Harris	37
	O. Gruenberger	43
	Elmer G. Hooper	44
	F. Theodore Mavis	46
	Mario Medici	51
	A. H. Blaisdell	63
	Morrough P. O'Brien	66
	R. T. Liddicoat	67
	Lloyd H. Donnell	71
	S. Timoshenko	72
	A. B. Cox	76
	Allen F. Sherzer	80
1698	**THE GREAT YANGTZE BAR.**	
	By **A. V. Hugo von Heidenstam**	102
	Discussion:	
	By A. W. Robinson	120
	R. L. Vaughn	124
1699	**RE-DESIGNING CATAWBA STATION FOR SERVICE ON A LARGE TRANSMISSION SYSTEM.**	
	By **W. S. Lee**	127
	Discussion:	
	By H. A. Hageman	145
	H. V. Joslin	148
	T. H. Hogg	149
	W. S. Lee	151
1700	**UNUSUAL ENGINEERING FEATURES OF AN IMMENSE THEATRE BUILDING.**	
	By **R. McC. Beanfield**	152
	Discussion:	
	By Theodore Belzner	182
	Fred A. Noetzli	182
	A. Floris	183
	Laurence J. Waller	187
	R. McC. Beanfield	188

No.		PAGE
1701	**ADVANCES IN WATERWAYS ENGINEERING DURING A HALF CENTURY.**	
	By W. M. Black	191
	Discussion:	
	By C. L. Hall	214
	DeWitt D. Barlow	214
	W. M. Black	215
1702	**THE DELAWARE RIVER FROM PHILADELPHIA TO THE SEA.**	
	By F. C. Boggs	218
	Discussion:	
	By Charles M. Spofford	246
	Henry J. Sherman	247
	Earl I. Brown	249
	Harry Taylor	253
	F. C. Warner	255
	William Gerig	258
	F. C. Boggs	259
1703	**THE HEAD-WORKS OF THE IMPERIAL CANAL.**	
	By C. E. Grunsky	262
	Discussion:	
	By E. S. Lindley	267
	C. E. Grunsky	269
1704	**THE SCIENCE OF FOUNDATIONS—ITS PRESENT AND FUTURE.**	
	By Charles Terzaghi	270
	Discussion:	
	By Charles R. Gow	302
	E. P. Goodrich	303
	Victor A. Endersby	304
	George Paaswell	305
	C. C. Williams	306
	Herbert Chatley	313
	J. Albert Holmes	314
	Jacob Feld	319
	W. S. Housel	322
	Frank S. Bailey	330
	T. L. Condron	331
	Lazarus White	333
	R. D. N. Simham	346
	Frank A. Marston	350
	Arthur M. Shaw	354
	D. P. Krynine	357
	S. P. Wing	365
	M. L. Enger	371
	Albert Jorgensen	378
	Charles Terzaghi	380
1705	**MAXIMUM FLOOD DISCHARGE IN SAN JOAQUIN VALLEY, CALIFORNIA.**	
	By Oren Reed	406

No.		PAGE
	Discussion:	
	By ROBERT FOLLANSBEE	431
	HAROLD K. FOX	431
	CHARLES H. LEE	433
	OREN REED	440
1706	**BAFFLE-PIER EXPERIMENTS ON MODELS OF PIT RIVER DAMS.**	
	By I. C. Steele and R. A. Monroe	451
	Discussion:	
	By EDWARD C. SHERMAN	481
	B. E. TORPEN	487
	OREN REED	497
	HAROLD K. FOX	498
	ROBERT F. EWALD	499
	F. W. SCHEIDENHELM	506
	RALPH L. PARSHALL	507
	H. W. DENNIS	511
	I. A. WINTER	516
	PAUL BAUMAN	518
	MORROUGH P. O'BRIEN	528
	J. C. STEVENS	530
	I. C. STEELE and R. A. MONROE	539
1707	**METHODS USED IN THE CONSTRUCTION OF TWELVE PRE-CAST CONCRETE SEGMENTS FOR THE ALAMEDA COUNTY, CALIFORNIA, ESTUARY SUBWAY.**	
	By Alvin A. Horwege	547
	Discussion:	
	By JOSEPH A. KITTS	565
1708	**THE VIRGINIAN RAILWAY ELECTRIFICATION.**	
	By George Gibbs	571
	Discussion:	
	By W. B. POTTER	628
	MARCEL JAPIOT	629
	L. K. SILLCOX	630
	I. OFVERHOLM	633
	RALPH BUDD	634
	GUIDO SEMENZA	634
	F. H. SHEPARD	635
	E. R. HILL	636
	NORMAN LITCHFIELD	638
	SIDNEY WITHINGTON	639
	GEORGE J. RAY	641
	JOHN P. HOGAN	642
	R. D. N. SIMHAM	644
	GEORGE GIBBS	648
1709	**FLOOD CONTROL WITH SPECIAL REFERENCE TO THE MISSISSIPPI RIVER: A SYMPOSIUM**	655
	INTRODUCTION.	
	By Edgar Jadwin	657

No. PAGE

RESUME OF THE MISSISSIPPI RIVER FLOOD PROBLEM.
By C. McD. Townsend 659

RAINFALL CHARACTERISTICS OF THE MISSISSIPPI DRAINAGE BASIN.
By H. C. Frankenfield 675

RUN-OFF CHARACTERISTICS OF THE MISSISSIPPI RIVER DRAINAGE BASIN.
By Nathan C. Grover 690

THE WORK OF THE MISSISSIPPI RIVER COMMISSION.
By C. W. Kutz 697

FOREST COVER AS A FACTOR IN FLOOD CONTROL.
By E. F. McCarthy 716

RESERVOIRS FOR MISSISSIPPI VALLEY FLOOD PROTECTION.
By William Kelly 724

THE BASIS OF THE CASE AGAINST RESERVOIRS FOR MISSISSIPPI FLOOD CONTROL.
By Arthur E. Morgan 737

LEVEES AS A MEANS OF FLOOD CONTROL FOR THE MISSISSIPPI RIVER.
By J. F. Coleman 755

IMPROVEMENT OF NAVIGATION IN RELATION TO FLOOD CONTROL.
By Stuart C. Godfrey 762

THE FLOOD PROBLEM OF NEW ORLEANS, LOUISIANA.
By Marcel Garsaud 782

THE RELIEF OUTLETS AND BY-PASSES OF THE SACRAMENTO VALLEY FLOOD-CONTROL PROJECT.
By C. E. Grunsky 791

RECLAMATION AS AFFECTING FLOOD CONTROL.
By Elwood Mead 812

POWER AS AFFECTING FLOOD CONTROL.
By F. W. Scheidenhelm 815

THE EFFECT OF AGRICULTURAL DRAINAGE UPON FLOOD RUN-OFF.
By Sherman M. Woodward and Floyd A. Nagler 821

Discussion:
By J. P. Snow 840
Frank B. Maltby 841

No.		PAGE
	MORRIS KNOWLES	845
	CHARLES H. PAUL	853
	F. G. JONAH	854
	C. S. JARVIS	858
	EDWARD H. SCHULZ	871
	Q. C. AYRES	878
	H. S. GLADFELTER	879
	BYRON E. WHITE	881
	W. G. PRICE	887
	OREN REED	890
	HAROLD C. FISKE	891
	CHARLES F. BROOKS	894
	WILLIAM GERIG	898
	C. H. EIFFERT	900
	SHERMAN M. WOODWARD	903
	W. J. SCHLICK	905
	S. L. MOYER	907
	WILLIAM M. HALL	919
	C. E. RAMSER	933
	T. KENNARD THOMSON	934
	W. E. ELAM	937
	R. W. DAVENPORT	947
	E. W. LANE	949
	C. McD. TOWNSEND	951
	H. C. FRANKENFIELD	952
	E. F. MCCARTHY	953
	WILLIAM KELLY	954
	ARTHUR E. MORGAN	957
	MARCEL GARSAUD	962
	C. E. GRUNSKY	963
	SHERMAN M. WOODWARD AND FLOYD A. NAGLER	964
1710	**THE CONOWINGO HYDRO-ELECTRIC DEVELOPMENT ON THE SUSQUEHANNA RIVER.**	
	By Alexander Wilson, 3d	970
	Discussion:	
	By WILLIAM C. L. EGLIN	979
	J. V. HOGAN	981
	A. W. CLARK	990
	H. A. HAGEMAN	993
	ALEXANDER WILSON, 3D	997
1711	**PRECISE WEIR MEASUREMENTS.**	
	By Ernest W. Schoder and Kenneth B. Turner	999
	Discussion:	
	By H. W. KING	1111
	FLOYD A. NAGLER	1114
	A. STREIFF	1119
	R. L. PARSHALL	1127
	W. S. PARDOE	1130
	R. E. BALLESTER	1136
	GARDNER S. WILLIAMS	1142
	TH. REHBOCK	1143
	ERIK G. W. LINDQUIST	1163
	CLEMENS HERSCHEL	1176
	ERNEST W. SCHODER	1179

No.		PAGE
1712	**ANALYSIS OF ARCH DAMS BY THE TRIAL LOAD METHOD.**	
	By **C. H. Howell and A. C. Jaquith**	1191
	Discussion:	
	By B. F. Jakobsen	1226
	George R. Rich	1229
	C. A. P. Turner	1233
	F. H. Constant	1241
	F. W. Hanna	1246
	L. J. Mensch	1250
	William Cain	1265
	Fred A. Noetzli	1270
	Fredrik Vogt	1272
	J. L. Savage and Ivan E. Houk	1289
	A. Floris	1296
	C. H. Howell	1306
1713	**HYDROSTATIC UPLIFT IN PERVIOUS SOILS.**	
	By **H. de B. Parsons**	1317
	Discussion:	
	By F. R. Harris	1333
	J. C. Meem	1334
	Eugene E. Halmos	1339
	T. R. Lawson	1340
	J. Vipond Davies	1341
	Lazarus White	1342
	T. Kennard Thomson	1343
	Daniel E. Moran	1344
	Alexander V. Gallogly	1344
	Allen Hazen	1345
	Charles Terzaghi	1346
	Frank S. Bailey	1353
	Charles R. Gow	1353
	E. L. Chandler	1356
	Martin J. McPike	1357
	Jacob Feld	1360
	J. Albert Holmes	1362
	E. W. Lane	1363
	H. de B. Parsons	1364
1714	**CONTINUOUS BEAMS OVER THREE SPANS.**	
	By **I. Oesterblom**	1367
	Discussion:	
	By Frank S. Bailey	1384
	Clyde T. Morris	1385
	A. Lincoln Hyde	1386
	Harold E. Wessman	1388
	Robert M. Wilder	1392
	W. F. Way	1394
	H. M. Hadley	1396
	T. F. Hickerson	1405
	John I. Parcel	1410
	R. A. Caughey	1410
	John B. Snethlage	1420
	T. D. Mylrea	1421
	I. Oesterblom	1422

No.		PAGE
1715	**THE O'SHAUGHNESSY DAM AND RESERVOIR.**	
	By **John H. Gregory, C. B. Hoover, and C. B. Cornell**	1428
	Discussion:	
	By J. Ernest Carman	1493
	B. F. Jakobsen	1494
	W. H. Dittoe and G. Gale Dixon	1495
	John H. Gregory, C. B. Hoover, and C. B. Cornell	1498
1716	**THE TREATMENT OF THE WATER SUPPLY OF THE CITY OF COLUMBUS, OHIO.**	
	By **Charles P. Hoover**	1505
	Discussion:	
	By F. H. Waring	1519
	George W. Fuller	1522
	Robert Spurr Weston	1524
	C. W. Foulk	1525
1717	**UPWARD PRESSURES UNDER DAMS: EXPERIMENTS BY THE UNITED STATES BUREAU OF RECLAMATION.**	
	By **Julian Hinds**	1527
	Discussion:	
	By E. L. Chandler	1551
	B. F. Jakobsen	1558
	Charles Terzaghi	1559
	J. C. Stevens	1563
	F. W. Hanna	1564
	Ivan E. Houk	1565
	C. H. Howell	1571
	P. Wilhelm Werner	1573
	H. de B. Parsons	1574
	E. W. Lane	1576
	C. E. Grunsky	1578
	Julian Hinds	1580
1718	**HYDRAULIC STUDIES AND OPERATING RESULTS ON THE MIAMI FLOOD CONTROL SYSTEM.**	
	By **C. H. Eiffert**	1583
	Discussion:	
	By C. S. Bennett	1602
	Morris Knowles	1605
	Walter M. Smith	1606
	B. F. Jakobsen	1607
	K. L. De Blois	1610
	E. S. Lindley	1612
	C. S. Jarvis	1614
	E. W. Lane	1615
	Charles H. Paul	1616
	C. H. Eiffert	1617
1719	**LOAD DISTRIBUTION IN HIGH ARCH DAMS.**	
	By **R. A. Sutherland**	1623

No.		PAGE
	Discussion:	
	By WILLIAM CAIN	1666
	B. F. JAKOBSEN	1668
	PAUL BAUMAN	1670
	A. FLORIS	1675
	FREDRIK VOGT	1676
	F. H. CONSTANT	1677
	R. A. SUTHERLAND	1678
1720	**SILTING OF THE LAKE AT AUSTIN, TEXAS.**	
	By **T. U. Taylor**	1681
	Discussion:	
	By L. M. LAWSON	1690
	C. H. EIFFERT	1690
	OREN REED	1691
	CHARLES SCHULTZ	1692
	JOHN B. HAWLEY	1693
	H. F. ROBINSON	1695
	JULIAN MONTGOMERY	1703
	KIRK BRYAN	1703
	J. C. STEVENS	1707
	P. A. WELTY	1708
	BANKS MCLAURIN	1713
	R. G. TYLER	1715
	E. C. H. BANTEL	1717
	R. F. WALTER	1719
	WILLIAM JENNER POWELL	1728
	R. I. MEEKER	1731
	T. U. TAYLOR	1733
1721	**ENGINEERING AND ENGINEERING EDUCATION: ADDRESS AT THE ANNUAL CONVENTION AT MILWAUKEE, WIS., JULY 10, 1929.**	
	By **Anson Marston**	1736

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	PAGE
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	PAGE
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NEW THEORY FOR THE CENTRIFUGAL PUMP*

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With Discussion by Messrs. J. W. MacMeeken, Thomson Kingsford, Sanford A. Moss, P. L. Reed, Max M. Munk, Elmo G. Harris, O. Gruenberger, Elmer G. Hooper, F. Theodore Mavis, Mario Medici, A. H. Blaisdell, Morrough P. O'Brien, R. T. Liddicoat, Lloyd H. Donnell, S. Timoshenko, A. B. Cox, and Allen F. Sherzer.

Synopsis

This paper presents an outline of a program of research and experiment on centrifugal pumps conducted in the Hydraulic Laboratory of the University of Michigan. This study has resulted in:

1.—The discovery of a number of errors in present accepted theories;
2.—The development of an entirely new theory; and
3.—The successful application of the new theory in increasing the efficiency of the centrifugal pump.

Introduction

It is a matter of common knowledge among those intimately connected with the design and manufacture of the centrifugal pump that most of the literature on this subject is of little assistance in their work. The actual design of a centrifugal pump is based largely on the known performance of similar pumps, and the designer determines empirically the various dimensions, constants, etc. An English writer on hydraulics, John Perry, makes the following significant statement:

* Presented at the meeting of October 5, 1927.

† Associate Prof., Mech. Eng., Univ. of Michigan; Cons. Hydro-Mech. Engr., Ann Arbor, Mich.

"In this subject of flowing water there are more misleading hypotheses, due to perverted ingenuity, than in almost any other; and unfortunately the logical conclusions drawn from these hypotheses when known to be untrue are said to be the statements of theory as opposed to practice."

A careful examination of the centrifugal pump theory will show, it is believed, that he has hit the nail squarely on the head.

Notation

In this paper, the following symbols are used:

1 or e	= point on entrance edges of runner vane.
2 or a	= point on discharge edges of runner vane.
r_1	= radial distance to Point 1.
r_2	= radial distance to Point 2.
V_2 or U_a	= peripheral speed of the runner at its outer diameter.
V_1 or U_e	= peripheral speed of the runner at its inner diameter.
C_2 or W_a	= absolute velocity of the water discharged from the runner.
C_1 or W_e	= absolute velocity of the water at entrance to the runner.
W_2 or V_a	= relative velocity of the water to the runner at discharge.
W_1 or V_e	= relative velocity of the water to the runner at entrance.
g	= acceleration due to gravity.
γ	= weight per unit volume of the liquid, in pounds.
H_{at}	= height of water column supported by atmospheric pressure.
D_3	= outer diameter of the vortex formed in the pump casing.
V_3	= peripheral speed of outer diameter of the vortex.
C_{p2}	= tangential component of C_2.
C_{p1}	= " " " C_1.
α_2 or δ_a	= angle between C_2 and V_2.
α_1 or δ_e	= " " C_1 and V_1.
β_2 or β_a	= " " W_2 and V_2.
β_1 or β_e	= " " W_1 and V_1.
ω	= angular velocity, in radians per second.
A_1, A_2, A_3	= areas of Orifices Nos. 1, 2, and 3. (See Fig. 8.)
H_1, H_2, H_3	= decreases in pressure head due to flow through Orifices Nos. 1, 2, and 3.

The Present Theory

From among many references setting forth the usual theory of the centrifugal pump, three are selected as typical:

(1) "Pumping Machinery", by A. M. Greene, Jr. His treatment is similar to "Die Zentrifugalpumpen", by Fritz Neumann, and is typical of the German method.

(2) "Centrifugal Pumps", by Lowenstein and Crissey.

(3) "Centrifugal Pumps", by R. L. Daugherty. This derivation of theory resembles that found in British texts.

A review of these and similar works indicates that the theoretical working equations may be derived, in general, from two distinct lines of approach. One of these seems to be held in especial favor by European writers, particu-

larly in Germany, and is based on the application of Bernoulli's theorem to the flow through the pump. The other is often found in works of British and American origin and is based on the laws of angular momentum.

Present literature on this subject as typified by these authorities appears to be incorrect both hydraulically and mathematically. Further, the fundamental laws of action of the centrifugal pump seem never to have been explained properly, and many misconceptions exist, even among those supposedly familiar with the hydraulics of these pumps.

Derivation by Application of Bernoulli's Theorem.—To simplify matters, the discussion will be confined to the derivation of Lowenstein and Crissey.* This corresponds closely with that given by Greene, and usually the same remarks may be applied to both.†

This derivation contains a most important and interesting error which contributes largely to the defects in present pump theory. One derivation reads, as follows:

$$h_a + \frac{W_a^2}{2g} = h_l + \frac{W_l^2}{2g} + p_l h_n \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (A)$$

This is merely the application of Bernoulli's theorem to the problem, since h_a is the pressure head in the fluid at Point a, and W_a is the absolute velocity of the water discharge from the impeller at Point a; while h_l is the pressure head at l, the entrance to the guide-vanes (a short distance from Point a), and $\frac{W_l^2}{2g}$ is the absolute velocity head at l. The term, $p_l h_n$, represents the hydraulic losses occasioned by the flow from Points a to l. This expression is Bernoulli's theorem in its usual form.

It is commonly considered that the total head at a is composed of a pressure head, h_a, and, in addition, a velocity head, $\frac{W_a^2}{2g}$. When the pump discharge is approximately zero, the pressure head, h_a, is very nearly $\frac{U_a^2}{2g}$ as is well known. Under the same conditions the relative velocity, V_a, becomes practically zero and, hence, the absolute velocity, W_a, approaches the peripheral speed of the impeller at a, or U_a. The velocity head is, therefore, $\frac{U_a^2}{2g}$, or nearly so. As will be shown, this theory would logically lead to a discrepancy between theory and practice of about 100 per cent.

Current Fallacies.—The usual explanation of this discrepancy indicates a serious misconception not only of the action of the centrifugal pump, but of one of the fundamental laws of hydraulics. The error is due to a misapplication of the well-known Bernoulli theorem.

When water flows in a pipe it is customary to say that the total head is obtained by measuring the pressure head by a piezometer at right angles to the flow, and adding the velocity head. This is a correct application of the theorem. The pressure and velocity exist as separate and distinct factors. However, water being set in rotation in a cylindrical vessel by the action of

* "Centrifugal Pumps," p. 7.

† See Appendix II for derivation in full.

a paddle or impeller, there is a pressure created by centrifugal force which is equal to $\frac{U_a^2}{2g}$ and which is part and parcel of the velocity itself. The total head may be regarded as either, but not both. The pressure exists merely because of the velocity in the circular path, but certainly not in addition to it.

It would be as logical to say that the "hammer", swung around by the athlete, has a total energy equal to the sum of the kinetic energy of the ball and the tension in the wire. Clearly the tension is not energy, but merely a force existing because of the tendency of the ball to fly off on a tangent. The energy is entirely kinetic. The act of letting go of the wire (destroying the tension) does not change any of the original energy, and the hammer flies off at a tangent with a velocity, U_a, which is the same as its former peripheral velocity. Therefore, the total energy is $\frac{U_a^2}{2g}$.

It is clear that the pump gives a head of only $\frac{U_a^2}{2g}$ at "shut off", instead of $\frac{U_a^2}{g}$ as required by all present theories, for the very simple reason that the energy imparted is only $\frac{U_a^2}{2g}$ and naturally the water can have no more energy than is given it by the impeller.

One of the reasons that might be advanced as to the cause of this error in the pump theory is the widespread failure to distinguish properly between pressure and energy. Pressure may measure potential energy and may serve to transmit energy, but it is not in itself energy any more than force is work.

Derivation by Principle of Angular Momentum.—In order better to understand this derivation, refer to Fig. 1. The value, $C_2 \cos \alpha_2$. represents the tangential component of the absolute velocity leaving the impeller, and $\frac{1}{g} C_2 \cos \alpha_2$ is the tangential momentum per second for each pound of water passing through the impeller. Momentum per second is force, and, therefore, $\frac{1}{g} C_2 \cos \alpha_2$ represents a force acting on the impeller in a direction opposite that of rotation. This force may be conveniently regarded as concentrated at Point 2. The moment of this force is $\frac{1}{g} C_2 \cos \alpha_2 r_2$, in which, r_2 is the radial distance from the center of the shaft to Point 2. In a similar manner, $\frac{1}{g} C_1 \cos \alpha_1$ represents a force concentrated at Point 1 which, if α_1 is less than 90°, will assist in rotating the impeller. The moment of this force is $\frac{1}{g} C_1 \cos \alpha_1 r_1$. The total turning moment on the wheel is the difference of these two moments, or,

$$\frac{1}{g} C_2 \cos \alpha_2 r_2 - \frac{1}{g} C_1 \cos \alpha_1 r_1$$

or,

$$\frac{1}{g}\,(C_2 \cos \alpha_2\, r_2 - C_1 \cos \alpha_1\, r_1)$$

If this torque be multiplied by the angular velocity, ω, the result is the work done by the impeller on the water, or,

$$\text{Work} = \frac{1}{g}\,(C_2 \cos \alpha_2\, r_2\, \omega - C_1 \cos \alpha_1\, r_1\, \omega)$$

but $\omega\, r_2 = V_2$ and $\omega\, r_1 = V_1$. Hence,

$$\text{Work, or head} = \frac{1}{g}\,(C_2\, V_2 \cos \alpha_2 - C_1\, V_1 \cos \alpha_1) \ldots\ldots\ldots (B)$$

This is exactly the equation obtained by the application of Bernoulli's theorem to the flow of water through the pump.*

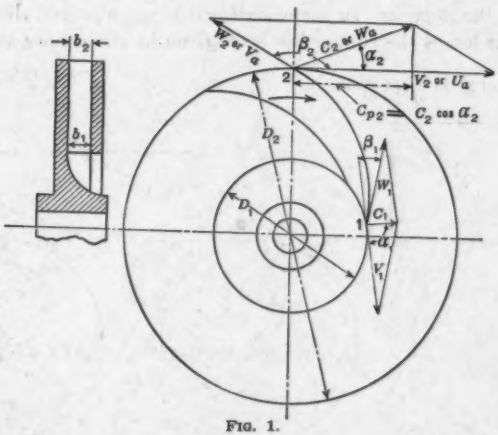

FIG. 1.

If the water approaches the vane at entrance in a radial direction, as is often assumed, obviously, the runner can receive no angular momentum from that source; hence,

$$C_p\, r_1 = 0$$

and,

$$H = \frac{1}{g}\left(C_{p2}\, r_2\right) = \frac{1}{g}\left(C_2\, V_2 \cos \alpha_2\right)^{\dagger} \ldots\ldots\ldots\ldots (C)$$

Evidently, as far as the final results go, one method is equivalent to the other.

Reasons for Observed Discrepancies.—Equations (*B*) and (*C*) form the basis of all present theories. They do not, and cannot, furnish a rational basis for the analysis and design of centrifugal pumps.

* See Equation No. 15, p. 12, in "Centrifugal Pumps," by Lowenstein and Crissey. Also, see Appendix II.

† This is identical with Equation No. 36, p. 62, in "Centrifugal Pumps," by Daugherty.

The reasons for this fallacy, as before, involve a misconception of the true action of a centrifugal pump. The increase in angular momentum of 1 lb. of water in passing through the impeller is,

$$\frac{1}{g}\,(C_{p2}\,V_2 - C_{p1}\,V_1)$$

On the assumption that the water approaches the entrance edge of the vanes in a radial direction and, hence, $C_{p1} = 0$, the work done on 1 lb. of water is found to be $\frac{1}{g}\,(C_{p2}\,V_2)$. This is nearly correct, but not for the reason given.

The water probably never approaches the vanes in a radial direction, but rotates with the impeller at all rates of flow. Hence, C_{p_1} is probably never zero. The error lies in the assumption that the water had some energy as it approached the runner which it received from some outside source. Naturally, if this is so, this should be deducted from the final energy to obtain that given by the impeller. In the centrifugal pump, however, all the energy in the water as it leaves the runner has been given by the runner and, hence, the increase in energy is $\frac{1}{g}\,(C_{p2}\,V_2)$.

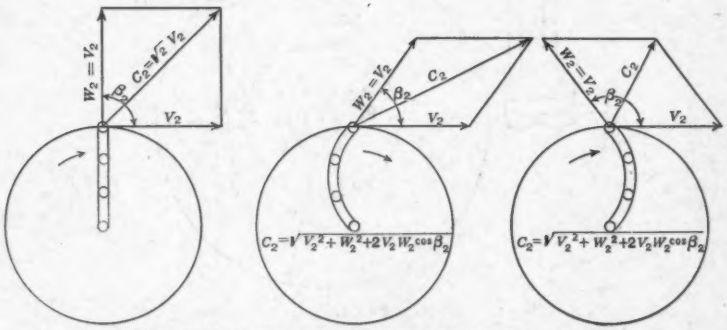

FIG. 2.—DISCHARGE DIAGRAMS FOR THE ASSUMED CONDITION OF FREE RADIAL ACCELERATION OF THE WATER.

NEW CONCEPTIONS OF PUMP ACTION

Imagine a circular disk with a radial groove from the center to the outer circumference (Fig. 2). Suppose that this disk revolves in a horizontal plane and that a very smooth round ball is introduced into the groove at the center. If there are no losses due to friction and the only forces acting are those of centrifugal force and inertia, the ball will cross the outer circumference in a radial direction relative to the moving disk with a velocity in that direction equal to its peripheral velocity, V_2*. The absolute velocity of the ball, therefore, will be the resultant of these two velocities, or $\sqrt{2} \times V_2$, in a direction inclined 45° to the tangent. The kinetic energy is clearly,

$$\frac{1}{2}\,M\,V_2^2, \text{ or } \frac{2\,V_2^2}{2\,g} = \frac{V_2^2}{g}$$

* See Appendix I for proof.

The energy given to the water, however, is only about one-half that obtained by this process. The reason is that the disk and ball and the centrifugal pump are by no means equivalent, as has been commonly supposed. For the similarity to hold, it would be necessary (1) that the vanes or grooves be radial; (2) that the particles be free to accelerate radially; and (3) that the particle be introduced at the center.

Real Action of Centrifugal Pump.—None of these conditions is fulfilled in the centrifugal pump. The water contained in the impeller rotates with it and owing to centrifugal force has a tendency to move out from the center. Outside the runner is the spiral casing filled with water. The pressure in this casing acts to retard the flow from the runner. If the discharge valve is opened the pressure falls in the casing and more water flows from the runner. In the same way if the discharge valve is closed the pressure rises in the casing, thus reducing the discharge.

Thus, the flow from the runner is due to the difference between the force tending to throw the water out (centrifugal force) and the pressure in the casing tending to hold the water in the runner. This, of course, entirely prevents the free radial acceleration necessary to obtain the theoretical $\frac{V_2^2}{g}$ so often given in texts. A runner without a casing and discharging into the air might approximate this condition, but such a machine would be of no practical importance in engineering. In addition, it would be impossible to introduce each particle of water at the center of the shaft.

The result, $\frac{V_2^2}{g}$, would obviously be true only for radial vanes, at least at the discharge tips, which is only one special case. The slot in the disk might be curved forward in the direction of rotation and hence discharge the ball with an absolute velocity greater than V_2 (Fig. 2), or it might be curved backward in the opposite direction, in which case the absolute velocity is less than V_2.* If these vanes were curved forward in such a manner as to be tangent to the outer circumference the final absolute velocity would be $2\ V_2$ in the direction of the tangent and the energy per pound would be,

$$\frac{4\ V_2^2}{2\ g} = 2\ \frac{V_2^2}{g}$$

On the other hand, if the vanes were curved backward and tangent, the absolute velocity and, hence, the kinetic energy, would be zero. Thus, the energy put into the ball may vary according to the curvature of the vanes from zero to $2\ \frac{V_2^2}{g}$, with a mean of $\frac{V_2^2}{g}$ for radial vanes. All this presupposes that the ball is free to accelerate radially, which is never true of the water in the pump. The fact that the present pump theory is purely kinetic and depends on the previous assumption of free radial acceleration seems to have been overlooked completely. This theory would be more applicable to a pump without a casing and discharging freely into the air. Under such conditions the

* See Appendix I.

energy given to the water is entirely kinetic and can be readily obtained from the vector diagrams.

Assuming free radial acceleration, the relative velocity at discharge, regardless of the direction of the vane (forward, radial, or backward), is always equal to the peripheral speed of the vane at its outer diameter, V_2. The only factor that could make it less would be friction in passing through the impeller. In such case the kinetic energy at exit is not the sole measure of the work done on the fluid, as that done in getting it to the outer circumference should also be included. This error is commonly made in drawing the vector diagram at the discharge from the impeller.

The implied loss due to friction in the impeller passages is the hidden assumption that causes much of the difficulty in fitting theory to practice. In Fig. 3 the correct and incorrect values of the relative velocities, W_2 and W_2', and of the absolute velocities, C_2 and C_2', are shown. If the fluid is free to accelerate radially without friction loss in the impeller, the relative velocity, W_2, must equal the peripheral speed, V_2. The direction of W_2 is that fixed by the vane angle, but its magnitude is constant and equal to V_2 under the assumptions. Actually, the value of W_2 is often much less than V_2, as W_2'; but in such cases the original assumptions are not realized.

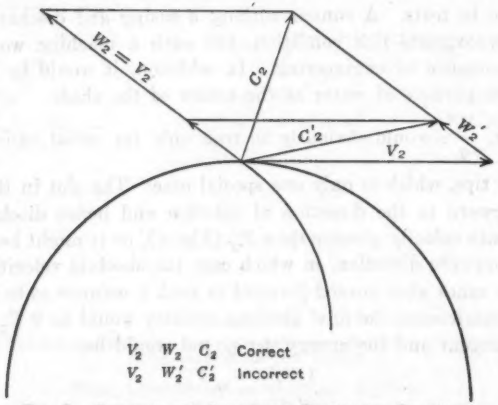

FIG. 3.—CORRECT AND INCORRECT DISCHARGE DIAGRAMS.

The restriction of the relative velocity, W_2, by the pressure in the casing is accomplished very efficiently, whereas an equal restriction caused by friction in the impeller passages would be very wasteful of energy. Similarly, the flow of water from a long pipe might be regulated very efficiently by a weir of variable height at the discharge end, but if the regulation was by a valve placed in the line, energy would be lost in friction.

EXPERIMENTAL VERIFICATION

Theoretical vs. Actual Heads.—In Fig. 4(*a*) is represented graphically the generally accepted theory of the centrifugal pump. Here the ideal quantity-head curve is shown beside the actual quantity-head curve for comparison.

Similar curves may be found in numerous works of reference.* There is an obvious inconsistency between the equations representing the head imparted to the water and their graphical representation as in Fig. 4(*a*). For example, the ideal quantity-head curve is seen to pass through a point representing more than double the head actually developed at the point of zero discharge—an obvious impossibility as the pressure is caused only by centrifugal force.

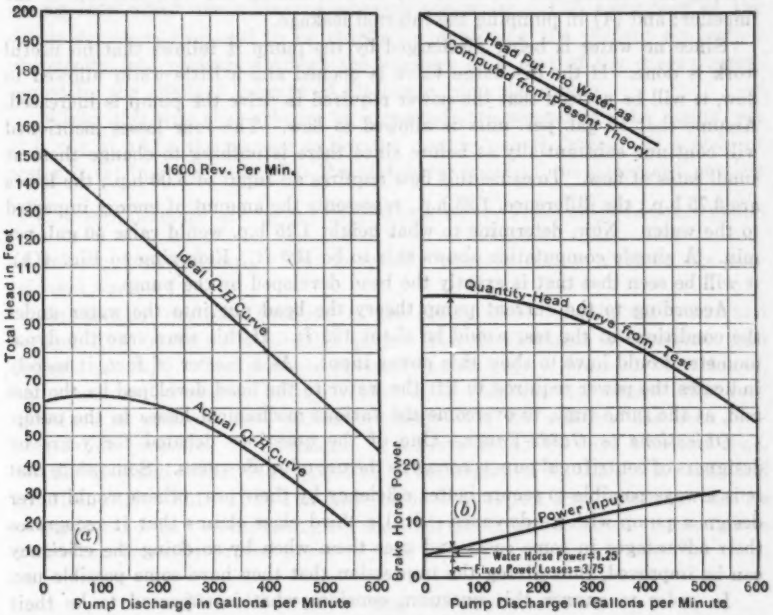

FIG. 4.—GRAPHICAL COMPARISON OF THEORETICAL AND ACTUAL HEADS IN CENTRIFUGAL PUMPS.

According to the present theory the total head is considered to be composed of two items, pressure and velocity. A measurement of the pressure indicates an average value of about $\frac{U_a^2}{2g}$ at or near the point of zero discharge. Discharge diagrams of runners show that at small rates of flow the absolute velocity, W_a, is nearly equal to U_a and, hence, the energy represented by it is nearly $\frac{U_a^2}{2g}$. This would apparently mean that the total head is approximately $\frac{U_a^2}{2g} + \frac{W_a^2}{2g}$, or very nearly $\frac{U_a^2}{g}$. As a matter of actual experience only a head of about one-half the ideal is developed by the pump. The difference is commonly said to be due to shock losses and many elaborate formulas have been developed to account for it.

* "Centrifugal Pumps," by R. L. Daugherty, pp. 76-77, Figs. 58-59.

Test Determination of Power Expenditure.—Fortunately, it is possible to settle this question beyond any doubt by a simple test in the laboratory. Referring to the characteristic curves obtained from a test of a centrifugal pump, as in Fig. 4(*b*), it will be noticed that at the point of zero discharge the power required to drive the pump is the minimum (3.75 h.p.). This power is consumed as follows: (1) In overcoming the bearing friction; (2) in overcoming the stuffing-box friction; (3) in overcoming the disk friction of the impeller; and (4) in pumping the internal leakage.

Since no water is being discharged by the pump it follows that no useful work is done. If the discharge valve is opened and a little water allowed to flow, it will be noticed that the power required to drive the pump is increased. Assume that 50 gal. per. min. is allowed to flow. The four losses mentioned will continue substantially as before since there is nothing to change them at small rates of flow. To cause this flow requires an input of 5.00 h.p.; the losses are 3.75 h.p.; the difference, 1.25 h.p., represents the amount of energy imparted to the water. Now, determine to what height 1.25 h.p. would raise 50 gal. per min. A simple computation shows this to be 100 ft. Referring to Fig. 4(*b*), it will be seen that that is exactly the head developed by the pump.

According to the current pump theory the head put into the water under the conditions of the test would be about 195 ft. If this were true the dynamometer would have to show this power input. As a matter of fact, it merely indicates the power required to lift the water to the head developed by the test and, at the same time, to overcome the various mechanical losses in the pump.

Objections to Guide-Vanes.—One of the questions debated for years by designers of centrifugal pumps concerns the use of guide-vanes. Some state that it is always possible to secure better efficiency by their use; others would never design a pump with guide-vanes. Still a third class claims that it recognizes their advantages in some cases and uses them when by so doing the efficiency can be improved, thus giving the impression that they have some possible use.

In order to answer this question, consider what is supposed to be their function. It has been shown in the various theories that at the discharge from the runner the water has a pressure head, $\frac{P_2}{\gamma}$, as well as a velocity head, $\frac{C_2^2}{2g}$. It is often stated that this velocity head, $\frac{C_2^2}{2g}$, will be lost in shock, friction, etc., unless suitable provision is made for its recovery. The commonly accepted idea has been that at very small rates of flow this energy is about equally divided between pressure and velocity and that the entire velocity head under these conditions was apt to be lost unless special provision was made in the form of guide-vanes or diffuser vanes. Such vanes were designed so as to receive the absolute velocity of the water. By increasing the area of the water passages between them, the velocity was expected to be converted most efficiently into pressure.

Evidently, from the previous discussion, the use and benefits of guide-vanes are founded on a misconception of the theory. In a centrifugal pump it is absolutely wrong to assume that the total energy is represented by the pressure in the casing plus the kinetic energy of the absolute velocity of discharge.

As the water is confined to a circular path, its kinetic energy is necessarily converted into pressure without any assistance from the guide-vanes. In fact, they often serve to obstruct the natural flow of water, which entirely defeats their purpose and renders them useless. From the point of view of practice equal or better efficiency can be obtained with a properly designed volute.

Purpose of Casing.—What has just been said with regard to the use of guide-vanes may also be extended to include the so-called volute or spiral casing into which the runner discharges the water. The advantage of the volute casing is supposed to lie chiefly in permitting the efficient conversion of velocity into pressure thrdugh the recovery of a part of the energy represented by the absolute velocity of discharge from the runner.

While it is true that velocity is converted into pressure in the casing, yet it is not at all in the manner or for the reasons usually given. If a body of water is set in rotation so that the peripheral speed of the outermost particle is V, then a pressure will be developed equal to $\frac{V^2}{2g}$, a case of perfect conversion of velocity into pressure. This is exactly what is taking place in the centrifugal pump.

Apparently, the only function of the casing is to restrain the water and compel it to move in a circular path. The development of pressure then follows naturally. The pressure obviously exists only because of the velocity, and one may be said to be equivalent to or measure the other. The very existence of pressure here indicates clearly that the velocity has been converted into pressure. It is difficult to see how it can be done a second time, yet this is exactly what is supposed to be accomplished by the guide-vanes and spiral casings. This shows how hopeless it is to expect any gain from the use of guide-vanes and spiral casings, since their advantage is based on conditions which can be easily proved never to have existed.

New Analysis of the Centrifugal Pump

The Impeller and Its Action.—A pump is fundamentally a device for increasing the energy of the fluid passing through it. In the centrifugal pump this energy is first in the kinetic, and, later, in the pressure form. The conversion of velocity into pressure takes place in the casing as will be shown. For the present, however, consider the action of the runner quite apart from that of the casing.

Imagine a runner arranged so as to receive the water at its suction entrance and discharge it into the air—a device resembling a lawn sprinkler.

A particle of water, initially at rest at the level of the source of supply, soon leaves the runner with high velocity and, therefore, possesses kinetic energy. All this energy must necessarily have come from the impeller. Numerically, it is equal to $\frac{C_2^2}{2g}$ per lb. of water, neglecting any slight static head which might be present in addition to the velocity head, $\frac{C_2^2}{2g}$. The

design of the vanes at the discharge from the runner determines the amount of energy given to the water. This can be seen by reference to Fig. 5. If the vanes are curved forward in the direction of rotation the kinetic energy given to the water will increase as the discharge from the pump is increased. On the other hand for backward curved vanes the reverse will be true.

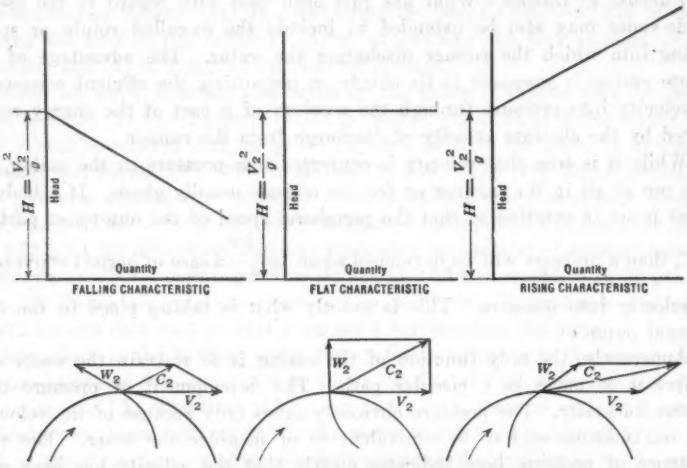

FIG. 5.—EFFECT OF THE VANE ANGLE, β_2, ON THE PUMP CHARACTERISTIC ACCORDING TO PRESENT THEORY.

In remarking on this, texts usually add that if the vanes are radial the energy given to the water is independent of the rate of discharge.* This is obviously incorrect as may be seen from Fig 5. All that can safely be said for such a condition is that the energy imparted is less affected by the rate of discharge. The relation between these elements is called the quantity-head curve, or often the pump characteristic (Fig. 5). In the case of forward curved vanes the head increases as the rate of discharge, and the runner is said to have a rising characteristic. If the vanes are radial it is usually (incorrectly) assumed that the head is independent of the discharge, and such runners are said to have a flat characteristic. When the vanes are curved backward the head decreases as the rate of flow, and the characteristic is said to be steep or falling.† All this, with some exceptions, would be substantially correct for a pump without a casing, but textbooks fail to state that qualification.

As a matter of fact in centrifugal pumps as built commercially the vane angle is only one of several factors that determine the pump characteristic. It does not, as many imagine, entirely control the shape of the quantity-head curve. If there were no casing and the water were free to discharge into the

* See "Centrifugal Pumping Machinery," by DeLaval, p. 58, Fig. 56.
† "Centrifugal Pumps," by R. L. Daugherty, p. 63, Fig. 55.

air the total energy imparted to the water could be measured by the work done on it in giving it its absolute kinetic energy, $\frac{C_2^2}{2g}$ per lb. of water.

Effects of Casing.—As a matter of fact pumps always have a casing, generally spiral in design, which completely encloses the impeller. The water in this casing between the runner and the side-walls and also in the volute is set in rotation by the impeller. The water discharged from the runner with an absolute velocity, C_2, has a relatively large tangential component. Mingling with that circling in the casing, it assists in maintaining the revolution. As the water is confined to a somewhat circular path a pressure is created in the casing due to centrifugal force.

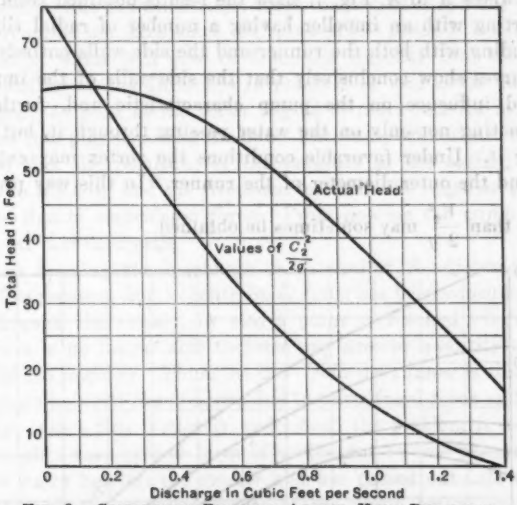

FIG. 6.—COMPARISON BETWEEN ACTUAL HEAD DEVELOPED AND THAT COMPUTED FROM VALUES OF THE ABSOLUTE VELOCITY OF DISCHARGE FROM THE RUNNER, OR C_2.

To use a very homely illustration this might be compared to a paddle stirring the water in a cylindrical tank assisted by a jet of water applied tangentially. The final velocity of rotation is the result of these two causes; but it is difficult to separate the effect of each. Basing the theoretical power input on the value of the absolute velocity of discharge from the runner (C_2) much less power is obtained than is actually supplied as determined by an accurate test. In Fig. 6 the value of $\frac{C_2^2}{2g}$ has been plotted as a function of the discharge. The actual quantity-head curve is also shown for comparison. Actually, much more energy has been imparted to the water than can be accounted for on the basis of the absolute velocity head, $\frac{C_2^2}{2g}$. It seems more

nearly to approach the constant value, $\frac{V_2^2}{2g}$, but probably is actually some intermediate value yet to be determined.

The runner acts not only on the water passing through it, but also on that outside, between it and the side-walls of the casing. The water in this space is set in rotation by the dragging action of the side-walls of the impeller. As a result, a vortex is formed within the case quite independent of whether or not the pump is actually discharging any water.

The nature of the side-walls of the impeller and case determines the extent to which this vortex may be formed. A number of experiments were performed to determine the effect of various conditions of the side-walls of the impeller. Curves *A* to *E*, Fig. 7, show the results obtained from a succession of tests starting with an impeller having a number of radial ribs cast on its sides and ending with both the runner and the side-walls entirely smooth.

These curves show conclusively that the side-walls of the impeller have a very marked influence on the pump characteristic and, further, that the impeller is acting not only on the water passing through it, but also on that surrounding it. Under favorable conditions the vortex may extend into the casing beyond the outer diameter of the runner. In this way pressures somewhat higher than $\frac{V_2^2}{2g}$ may sometimes be obtained.

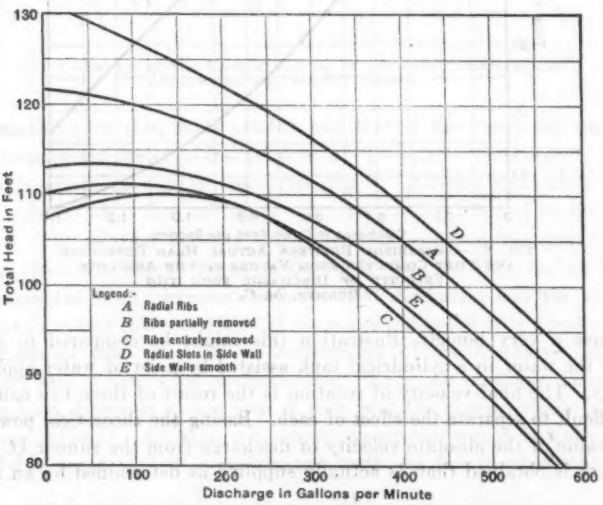

FIG. 7.—CHARACTERISTIC CURVES OF A 4-INCH CENTRIFUGAL PUMP AT 1 750 REVOLUTIONS PER MINUTE, SHOWING VARIATION IN THE CHARACTERISTIC CAUSED BY DIFFERENT SIDE-WALLS OF IMPELLER.

The absolute velocity of the water as it leaves the impeller will assist in maintaining this rotation and the vortex results from their combined action. The outer diameter of this vortex, which determines the pressure developed by the pump, will be called D_3 and the peripheral speed of the water at that

point, V_3. It is likely that D_3 and, hence, V_3 will vary somewhat with the pump discharge. Owing to the fact that the water is confined to a circular path, this velocity, V_3, is converted into pressure and causes the water to rise to a height, $\frac{V_3^2}{2g}$.

Action After Flow Begins.—Consider a pump running at a certain speed, but with its discharge valve closed. The water in the impeller and the casing is set in rotation, creating a pressure in the pump, but flow from the runner is prevented owing to the pressure of the water in the casing. Now, assume that the discharge valve is opened a trifle to permit a slight amount of flow. The pressure in the casing falls, and the pressure opposing the flow from the impeller decreases, but the centrifugal force tending to throw the water out of the impeller remains the same since the speed of the pump is constant.

Consequently, water will flow in from the impeller into the casing, the quantity depending on the difference in pressure as well as on the area of the water passages. As soon as the water begins to move relative to the impeller friction comes into play opposing the flow. The pump will adjust itself to these conditions so that the resultant of the pressure in the casing, the drop in pressure due to the flow, and the frictional losses in the impeller will equal the total pressure due to centrifugal force. From now on the pump will operate continuously at this discharge.

On further opening the discharge valve similar re-adjustments continue until the entire pressure due to centrifugal force has been consumed in forcing the water through the runner, or else a point is reached where the atmospheric pressure is no longer able to force any greater quantity of water into the suction of the impeller. Thus, the flow from the runner is simply governed by the difference between the pressure due to centrifugal force and the pressure in the casing. Once this is clearly understood the problem is merely that of the flow through a very special form of orifice due to this pressure difference.

After the water has left the runner and has passed out into the casing, it must flow through the so-called throat of the pump before passing the discharge flange. This throat is merely the point of maximum cross-section of the volute. All the water discharged from the runner passes through this, a third orifice.

Pump Considered as Three Orifices.—Hence, the entire problem of the centrifugal pump is that of flow through three orifices in series. The first is at the suction entrance to the impeller; and this determines the maximum capacity of the pump. The second is at the discharge from the runner, and, the third is at the throat of the pump. Each of these orifices or points of restriction has its own characteristics and their combined effect determines the pump characteristic.

Consider that the pump is running at a constant speed. The pressure developed by centrifugal force, tending to force the water out of the impeller, is constant and is quite independent of the flow or the amount of it. Centrifugal force is equal to $\frac{M V^2}{r}$; evidently, neither V nor r is greatly changed

by the flow of water and the mass, M, need not be considered in determining the energy input per pound of water. In other words, if there were no loss of pressure in the impeller and casing, the pump would deliver any quantity of water within the limit of the suction capacity at a constant pressure of approximately $\frac{V_2^2}{2g}$. A drop in pressure at the discharge flange is necessary if any water is to flow from the impeller.

The quantity that will flow depends on the area of the water passages, and its velocity, on the pressure difference established. The velocity is, therefore, quite independent of the quantity being discharged. The quantity is the product of the velocity and the area of the water passages measured perpendicularly to it.

The amount the pressure at any point falls below the horizontal line tangent to the quantity-head curve at "shut-off" is very nearly that required to force this quantity of water through the pump. The total drop in pressure is (1) that required to cause the given quantity of water to flow; and (2) that required to overcome the hydraulic losses. These are fundamentally different in their nature and yet produce similar effects on the pump characteristic.

Analogy Between Pump and Electric Generator.—The drop in pressure required to produce flow is an example of a loss in pressure without any loss in power. A similar instance occurs in the field of electricity. An electric generator would deliver a constant voltage at its terminals were it not for two principal losses. These are the drops in pressure due to armature reaction and to internal resistance. The drop in pressure due to armature reaction causes no loss of energy, whereas that due to resistance wastes energy in the form of heat. A loss of pressure that does not involve a loss of power is called a pressure drop, while a loss of power is called a pressure loss. These same terms might apply to the pump theory. In fact, many close analogies exist between the centrifugal pump and a separately excited shunt generator.

In the pump the drop in pressure without loss of power merely means that a greater pressure drop is required to discharge a given quantity of water through a small orifice than for one of larger area; or that for a given pressure drop, more water will flow through a large orifice than a small one. This does not mean that the large orifice is necessarily more efficient than the smaller one for both may have a coefficient of nearly 100 per cent.

When water flows through the runner it has to overcome the frictional losses. These produce a drop in pressure, which is also a loss of power. Since the length of the path of the water in the impeller is short and the water ways are smooth, this loss of power due to friction usually is not large.

Application of Theory to Determine Pump Characteristic

To make the new theory clear and indicate its application to practice, three characteristic curves, showing the relation between the capacity and

head, at constant speed, have been worked out for different typical pumps. These machines are of usual design and construction and of well-known makes. Fig. 8, showing typical views, serves to illustrate the three points of control or orifices. The first is at the entrance to the impeller-vane passages, and its area is equal to the product of the dimensions, B and D, and the total number of vane passages. The second is at the discharge from the impeller-vane passages; its area is the product of the dimensions, A and C, and the total number of vane passages. The third is at the throat of the pump; its area is usually that of a circle of diameter, E (Fig. 8).

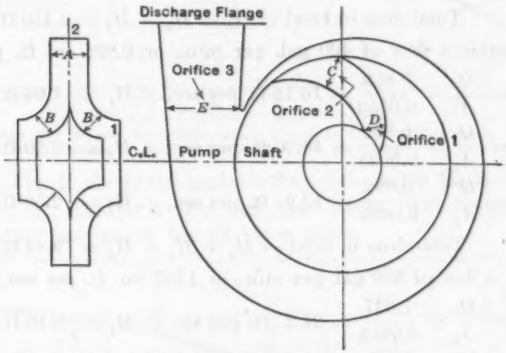

FIG 8.—VIEWS OF SHELL AND RUNNER, SHOWING LOCATION OF THE THREE ORIFICES.

Analysis of the Characteristic of a 4-Inch Double-Suction Centrifugal Pump.—For Orifice 1 at entrance to the impeller vanes:

$$\text{Area } A_1 = B \times D \times \text{number of vanes} \times 2 = 0.0553 \text{ sq. ft.}$$

For Orifice 2 at discharge from impeller:

$$\text{Area } A_2 = A \times C \times \text{number of vanes} = 0.0625 \text{ sq. ft.}$$

For Orifice 3 at the throat of the shell:

$$\text{Area } A_3 = \frac{\pi E^2}{4} = 0.0694 \text{ sq. ft.}$$

The velocity through the orifice is equal to $\frac{Q}{A}$, hence the velocity through Orifice 1 is,

$$\frac{Q}{A} = \frac{Q}{0.0553} = C_1 \text{ ft. per sec.}$$

the velocity through Orifice 2 is,

$$\frac{Q}{A_2} = \frac{Q}{0.0625} = C_2 \text{ ft. per sec.}$$

and the velocity through Orifice 3 is,

$$\frac{Q}{A_3} = \frac{Q}{0.0694} = C_3 \text{ ft. per sec.}$$

The drop in head or pressure required to produce the speed, C_1, C_2 or C_3, can be computed from the well-known formula, $H = \frac{C^2}{2g}$.

If a flow of 200 gal. per min. (= 0.446 cu. ft. per sec.) be assumed:

$$C_1 = \frac{Q}{A_1} = \frac{0.446}{0.0553} = 8.05 \text{ ft. per sec.} \therefore H_1 = 1.010\text{-ft. drop}$$

$$C_2 = \frac{Q}{A_2} = \frac{0.446}{0.0625} = 7.13 \text{ ft. per sec.} \therefore H_2 = 0.765\text{-ft. drop}$$

$$C_3 = \frac{Q}{A_3} = \frac{0.446}{0.0694} = 6.43 \text{ ft. per sec.} \therefore H_3 = 0.645\text{-ft. drop}$$

Total drop in head $= H_1 + H_2 + H_3 = 2.410$ ft.

Next, assume a flow of 400 gal. per min., or 0.892 cu. ft. per sec.:

$$C_1 = \frac{Q}{A_1} = \frac{0.892}{0.0553} = 16.15 \text{ ft. per sec.} \therefore H_1 = 4.06\text{-ft. drop}$$

$$C_2 = \frac{Q}{A_2} = \frac{0.892}{0.0625} = 14.30 \text{ ft. per sec.} \therefore H_2 = 3.19\text{-ft. drop}$$

$$C_3 = \frac{Q}{A_3} = \frac{0.892}{0.0694} = 12.9 \text{ ft. per sec.} \therefore H_3 = 2.59\text{-ft. drop}$$

Total drop in head $= H_1 + H_2 + H_3 = 9.84$ ft.

Next, for a flow of 600 gal. per min., or 1.337 cu. ft. per sec.:

$$C_1 = \frac{Q}{A_1} = \frac{1.337}{0.0553} = 24.2 \text{ ft. per sec.} \therefore H_1 = 9.10\text{-ft. drop}$$

$$C_2 = \frac{Q}{A_2} = \frac{1.337}{0.0625} = 21.4 \text{ ft. per sec.} \therefore H_2 = 7.10\text{-ft. drop}$$

$$C_3 = \frac{Q}{A_3} = \frac{1.337}{0.0694} = 19.3 \text{ ft. per sec.} \therefore H_3 = 5.80\text{-ft. drop}$$

Total drop in head $= H_1 + H_2 + H_3 = 22.00$ ft.

Finally, for a flow of 800 gal. per min., or 1.780 ft. per sec:

$$C_1 = \frac{Q}{A_1} = \frac{1.780}{0.0553} = 32.2 \text{ ft. per sec.} \therefore H_1 = 16.10\text{-ft. drop}$$

$$C_2 = \frac{Q}{A_2} = \frac{1.780}{0.0625} = 28.5 \text{ ft. per sec.} \therefore H_2 = 12.60\text{-ft. drop}$$

$$C_3 = \frac{Q}{A_3} = \frac{1.780}{0.0694} = 25.7 \text{ ft. per sec.} \therefore H_3 = 10.30\text{-ft. drop}$$

Total drop in head $= H_1 + H_2 + H_3 = 39.00$ ft.

The head developed by this pump at "shut off", or the point of no delivery, is 137 ft. as found by a test.

For various rates of flow the head is obtained by deducting the total drop, as calculated for any assumed rate of flow, from the shut-off head of 137 ft. Hence,

The head at 200 gal. per min. = 137.0 — 2.41 = 134.6 ft.
" " " 400 gal. per min. = 137 — 9.84 = 127.2 ft.
" " " 600 gal. per min. = 137 — 22.00 = 115.0 ft.
" " " 800 gal. per min. = 137 — 39.00 = 98.0 ft.

This is shown graphically in Fig. 9 by the full line. The actual quantity-head curve, as obtained from the test (dash line), is shown for comparison. Evidently, the two are in substantial agreement. The slight differences may be due to the fact that the dimensions used in the calculations were obtained from the design drawings and not from actual pump measurements.

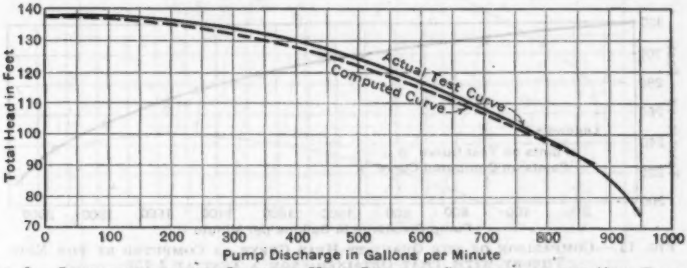

FIG. 9.—COMPARISON OF THE QUANTITY-HEAD CURVE AS COMPUTED BY THE NEW THEORY WITH THAT OBTAINED FROM A TEST AT 1 750 REVOLUTIONS PER MINUTE.

Similarly, Fig. 10 shows the analysis for a 3-in. pump for low or moderate heads and Fig. 11, that for a 6-in. high-head pump. The method is seen to yield satisfactory agreement for all three types.

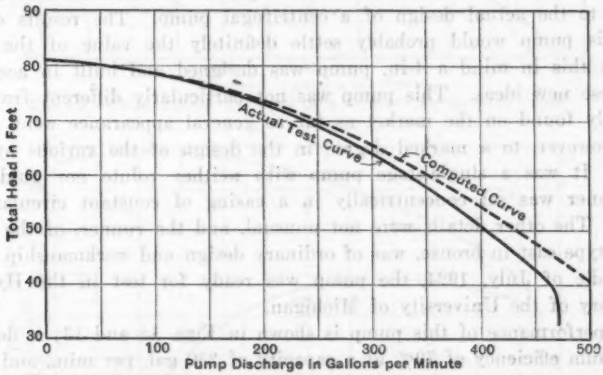

FIG. 10.—COMPARISON OF THE QUANTITY-HEAD CURVE AS COMPUTED BY THE NEW THEORY WITH THAT OBTAINED FROM A TEST AT 1 750 REVOLUTIONS PER MINUTE.

This should make clear the complete action of the centrifugal pump. It is then easy to explain the general effect of various changes in the design of a pump and to calculate the approximate numerical or quantitative effect of such changes on the pump characteristic. For obvious reasons this was not always possible with the previous theory.

APPLICATION OF NEW THEORY TO DESIGN

It would be a logical question whether this new theory is likely to result in any radical changes in the design or construction of centrifugal pumps. The fallacy on which the use of both the guide-vanes, or diffusers, and spiral casings, or volutes, is based, has already been mentioned. The new

theory indicates that all these accessories are not only useless but possibly even wrong. If this is correct it should be possible to obtain just as good results with a runner mounted concentrically in a plain circular casing of constant cross-section as with the most elaborately calculated spiral casings, volutes, diffusers, etc.

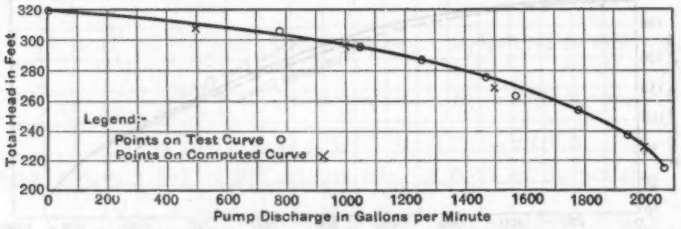

FIG. 11.—COMPARISON OF THE QUANTITY-HEAD CURVE AS COMPUTED BY THE NEW THEORY WITH THAT OBTAINED FROM A TEST AT 1 750 REVOLUTIONS PER MINUTE.

Results of First Design.—As this conclusion was a very radical departure from accepted current views it was felt that the final proof needed was to apply these principles (and others developed in connection with the new theory) to the actual design of a centrifugal pump. The results obtained with this pump would probably settle definitely the value of the theory.

With this in mind a 4-in. pump was designed and built in accordance with these new ideas. This pump was not particularly different from those ordinarily found on the market as far as general appearance went. It did differ, however, to a marked degree in the design of the various hydraulic details. It was a single-stage pump with neither volute nor guide-vanes. The runner was set concentrically in a casing of constant circular cross-section. The other details were not unusual, and the runner, of the double-suction type cast in bronze, was of ordinary design and workmanship. About the middle of July, 1924, the pump was ready for test in the Hydraulic Laboratory of the University of Michigan.

The performance of this pump is shown in Figs. 12 and 13; it developed a maximum efficiency of 79% at a capacity of 330 gal. per min., and a head of 67 ft. at 1 450 rev. per min. It is believed that this establishes a record for both American and European practice for a pump of such capacity.

The extreme flatness of the efficiency curve in Fig. 13 should be noted, also the broadness of the equal efficiency curves in Fig. 12, This indicates that it has been possible to develop extremely high efficiencies without any sacrifice of other desirable features.

Further Verification.—Those engaged in the design, manufacture, and sale of centrifugal pumps are likely to demand proof of an experimental nature rather than mathematical deductions no matter how skillfully and carefully they may be drawn. The writer, therefore, has made every effort to include actual test results in conjunction with his previous more or less mathematical conclusions.

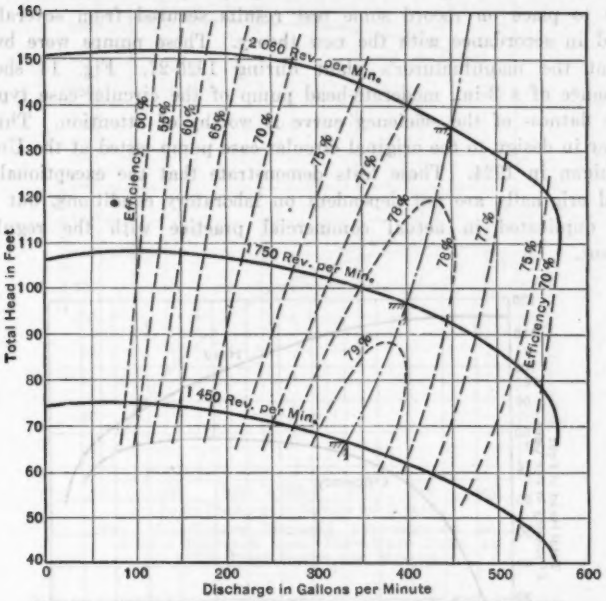

FIG. 12.—CHARACTERISTIC CURVE OF A 4-INCH CENTRIFUGAL PUMP.

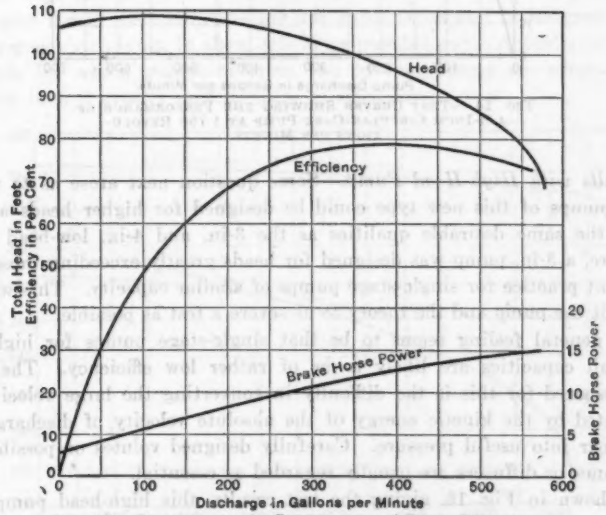

FIG. 13.—CHARACTERISTIC CURVES OF A 4-INCH CENTRIFUGAL PUMP AT 1 750 REVOLUTIONS PER MINUTE.

Through the courtesy of one of the large pump manufacturers, it is now possible to place on record some test results secured from several pumps designed in accordance with the new theory. These pumps were built and tested at the manufacturer's plant during 1925-27. Fig. 14 shows the performance of a 3-in., moderate-head pump of the circular-case type. The extreme flatness of the efficiency curve is worthy of attention. This pump is similar in design to the original circular-case pump tested at the University of Michigan in 1924. These tests demonstrate that the exceptional results obtained originally are not dependent on laboratory conditions, but may be readily duplicated in actual commercial practice with the regular test equipment.

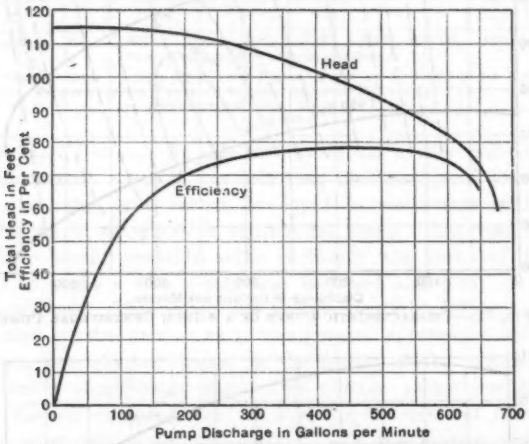

FIG. 14.—TEST CURVES SHOWING THE PERFORMANCE OF A 3-INCH CIRCULAR-CASE PUMP AT 1 750 REVOLUTIONS PER MINUTE.

Results with High-Head Pump.—Some question next arose as to whether or not pumps of this new type could be designed for higher heads and still exhibit the same desirable qualities as the 3-in. and 4-in. low-head pumps. Therefore, a 3-in. pump was designed for heads greatly exceeding those found in present practice for single-stage pumps of similar capacity. This was done to submit the pump and the theory to as severe a test as possible.

The general feeling seems to be that single-stage pumps for high heads and small capacities are likely to be of rather low efficiency. The reason often assigned for this is the difficulty in converting the large velocity head represented by the kinetic energy of the absolute velocity of discharge from the runner into useful pressure. Carefully designed volutes or possibly even guide-vanes or diffusers are usually regarded as essential.

As shown in Fig. 15, giving the test results, this high-head pump developed its maximum efficiency (72%) at a capacity of about 550 gal. per min. against a total head of 260 ft., at 1 750 rev. per min.

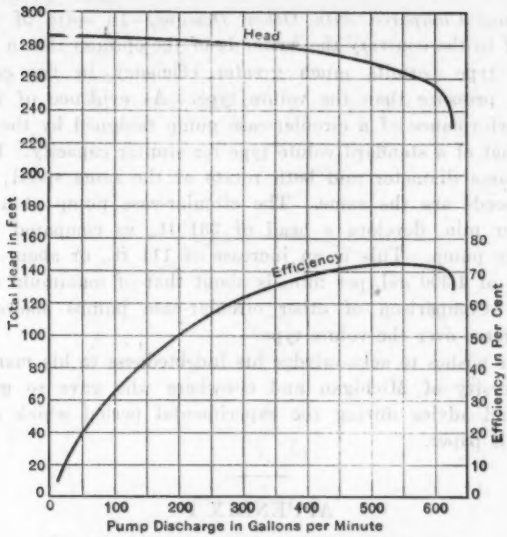

FIG. 15.—CURVES SHOWING PERFORMANCE OF A 3-INCH CIRCULAR CASE HIGH-HEAD PUMP AT 1 750 REVOLUTIONS PER MINUTE.

From this it would appear that the theory and design hold equally well for both high and low heads. It is interesting to note that the pump based on the new theory has neither volute nor guide-vanes and is designed, according to present standards, in about the worst possible way. Actually, it is doing the work for which pumps of from two to four stages are often required and even then it greatly exceeds them in efficiency.

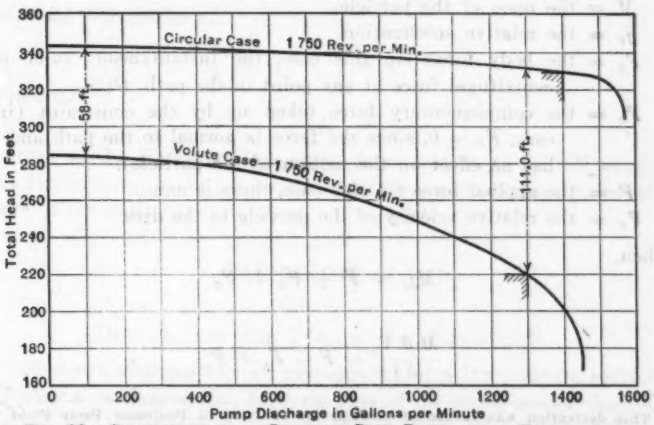

FIG. 16.—COMPARISON OF A CIRCULAR CASE PUMP AND A VOLUTE CASE PUMP OF THE SAME DIAMETER, SHOWING EXCESS HEAD DEVELOPED BY CIRCULAR CASE PUMP.

Performance Compared with Other Designs.—In spite of the probable general belief to the contrary the writer is of the opinion that a pump of the circular-case type permits much greater efficiency in the conversion of velocity into pressure than the volute type. As evidence of this, Fig. 16 shows the performance of a circular-case pump designed by the writer compared with that of a standard volute type for similar capacity. Both runners are of the same diameter and both rotate at the same speed; that is, the peripheral speeds are the same. The circular-case pump at a capacity of 1 300 gal. per min. develops a head of 331 ft., as compared with 220 ft. for the volute pump. This is an increase of 111 ft., or about 50 per cent. The capacity of 1 300 gal. per min. is about that of maximum efficiency for each pump. Comparison of other circular-case pumps showed somewhat the same increase over the volute type.

The writer wishes to acknowledge his indebtedness to his many colleagues at the University of Michigan and elsewhere who gave so generously of their time and advice during the experimental period which preceded the writing of this paper.

APPENDIX I*

Acceleration Caused by Rotation

Given a rotating disk of radius, r, turning at an angular velocity, ω. A particle is constrained to slide along a groove from the center, O, to a point, C, on the outer circumference. It is required to determine the relative velocity of the particle with respect to the disk, assuming that the particle starts from rest at the center, O.

From the general theory,† if,

M = the mass of the particle.

j_r = the relative acceleration.

F_b = the body force (in this case, the instantaneous value of the centrifugal force at any point in the path, OC).

F_c = the complementary force taken up by the constraint (in this case, $F_c = 0$, since the force is normal to the path and hence has no effect on the motion of the particle).

F = the applied force (in this case, there is none).

V_r = the relative velocity of the particle to the disk.

Then,

$$Mj_r = F + F_b + F_c$$

or,

$$\frac{M\,d\,V_r}{d\,t} = F + F_b + F_c$$

* This derivation was furnished through the courtesy of Professor Peter Field, of the College of Engineering, University of Michigan.

† "Introduction to Analytic Mechanics," by Ziwet and Field, p. 355.

Multiply by V_r; then,

$$M V_r \frac{d V_r}{d t} = (F + F_b + F_c) V_r$$

Multiply by $d t$; then

$$M V_r d V_r = (F + F_b + F_c) d r$$

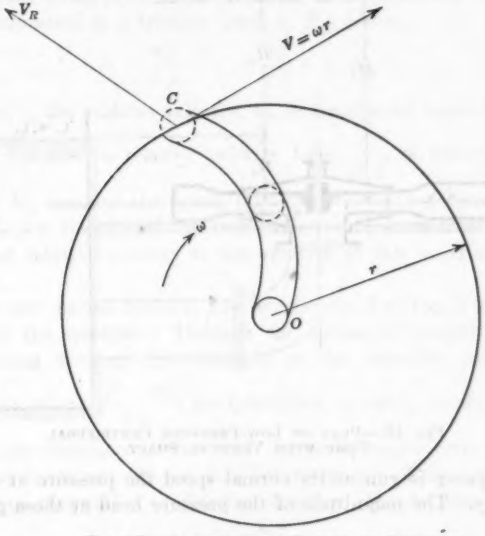

FIG. 17.—ACCELERATION CAUSED BY ROTATION.

But F and F_c are zero in this case; hence,

$$M V_r d V_r = \frac{M \omega^2 r^2 d r}{r}$$

This is the differential of,

$$\frac{1}{2} M V_r^2 = \frac{1}{2} M \omega^2 r^2$$

Therefore,

$$V_r^2 = \omega^2 r^2; \text{ or, } V_r = \omega r$$

In other words, if the particle starts from rest at O, it will accelerate to a relative velocity, $V_r = \omega r$, at the outer circumference, regardless of the curvature of the path.

APPENDIX II

DERIVATION OF THE FUNDAMENTAL EQUATIONS*

Fig. 18 shows schematically an arrangement of a low-pressure centrifugal pump having a vertical shaft. In every part of the pump proper there exists

* Reproduced substantially from "Centrifugal Pumps," by Lowenstein and Crissey.

a pressure greater than atmospheric, due to its depth below the level of supply. Imagine small pipes connected to the inside of the pump at points *s*, *e*, *a*, *l*, and *d*. As all these points lie in a horizontal plane, they have the same pressure head, h_a, when the pump is standing still.

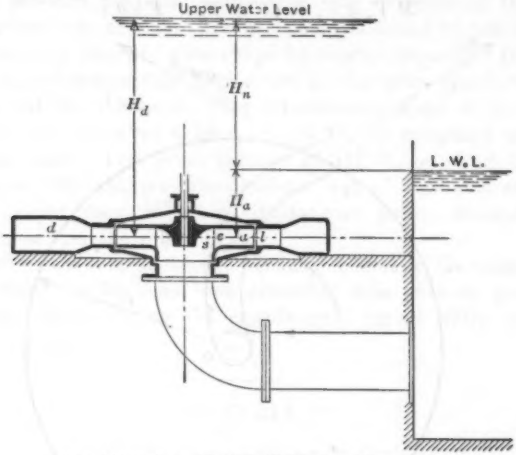

FIG. 18.—PLAN OF LOW-PRESSURE CENTRIFUGAL PUMP WITH VERTICAL SHAFT.

When the pump is run at its normal speed the pressure at each of these points will vary. The magnitude of the pressure head at these points is to be found.

The following derivation of the so-called "condition equation" is an old method used for determining the fundamental equation of water turbines. It gives a clear conception of the pressure and velocity distribution in the interior of a centrifugal pump. The pump shown in Fig. 18 is placed below the level of the supply water in order that all the pressure heads may be positive, that is, so that no suction exists. Of course the equations can be derived also for a pump placed above the level of the water.

When the pump is running there will be a pressure head, h_s, in the pipe at Point *s*. When the pump is at rest, it is less by an amount equal to a velocity head, $\frac{w_s^2}{2g}$, plus a friction head, $p_s H_n$. The friction head is dissipated in forcing the water through the entrance pipe up to Point *s*. The velocity in the entrance pipe, or suction tube, is designated by w_s, and the velocity head, $\frac{w_s^2}{2g}$, represents the loss of pressure head necessary to obtain the velocity, w_s, in the pipe.

Let H_n be the effective pressure or desired head of the pump. The condition equation may now be expressed by,

$$h_w = h_s + \frac{w_s^2}{2g} + p_s H_n \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

The next point, *e*, is taken directly at the entrance to the impeller. At this point a pressure head, h_e, and a velocity head, $\frac{w_e^2}{2g}$, exist, w_e being the absolute velocity of the water at this point. In the water flowing fr... Point *s* to Point *e* shock losses occur at the entrance to the channels of the impeller, which can be expressed as a friction head, $p_e H_n$, hence,

$$h_s + \frac{w_s^2}{2g} = h_e + \frac{w_e^2}{2g} + p_e H_n \quad \ldots\ldots\ldots\ldots\ldots\ldots (2)$$

If, at Point *e*, the relative velocity, v_e, is considered instead of the absolute velocity, w_e, then a relative velocity head, $\frac{v_e^2}{2g}$, is obtained, while the pressure head, h_e, remains the same. It is understood, of course, that absolute velocity is the velocity of the water relative to some fixed or stationary point, and that relative velocity is the velocity of the water relative to the rotating parts.

The water now passes through the impeller to Point *a*, which is directly at the exit of the impeller. Through the action of centrifugal force the water, in flowing through the channels of the impeller, has its pressure increased by the value, $\frac{u_a^2 - u_e^2}{2g}$, the quantities, u_a and u_e, being the peripheral velocities of the impeller at exit and entrance, respectively. Due to the friction of the water against the impeller walls and of the individual water particles among themselves, a friction head, $p_a H_n$, is lost. At Point *a*, a pressure head, h_a, and a velocity head, $\frac{v_a^2}{2g}$, exist, v_a being the relative exit velocity. Therefore, Equation (3) may be written:

$$\frac{v_e^2}{2g} + \frac{u_a^2 - u_e^2}{2g} = h_a + \frac{v_a^2}{2g} + p_a H_n \quad \ldots\ldots\ldots\ldots\ldots\ldots (3)$$

At Point *l*, which is directly at entrance to the guide-vanes, there exists a pressure head, h_l, and also a velocity head, $\frac{w_l^2}{2g}$. Due to shock losses at the entrance to the guide-vanes a friction head, $p_l H_n$, is lost in passing from Point *a* to Point *l*. At Point *a* the pressure head, h_a, and the velocity head, $\frac{w_a^2}{2g}$, exist, w_a being the absolute exit velocity. Hence,

$$h_a + \frac{w_a^2}{2g} = h_l + \frac{w_l^2}{2g} + p_l H_n \quad \ldots\ldots\ldots\ldots\ldots\ldots (4)$$

In order to convert as much as possible of the velocity head, $\frac{w_l^2}{2g}$, into pressure, the velocity, w_l, should be gradually reduced in passing from Point *l* through the guide-vanes. Let w_d represent the velocity and $\frac{w_d^2}{2g}$ the velocity head at the exit from the casing, and let $p_d H_n$ be the friction head expended

in carrying the water through the guide-passages and casing. Then,

$$h_l + \frac{w_l^2}{2g} = h_d + \frac{w_d^2}{2g} + p_d H_n \quad \ldots\ldots(5)$$

The first form of the so-called fundamental equation is obtained by adding Equation (1) to Equation (5), which gives Equation (6). For convenience Equations (1) to (5) are here re-stated:

$$h_w = h_s + \frac{w_s}{2g} + p_s H_n$$

$$h_s + \frac{w_s^2}{2g} = h_e + \frac{w_e^2}{2g} + p_e H_n$$

$$h_e + \frac{v_e^2}{2g} + \frac{u_a^2 - u_e^2}{2g} = h_a + \frac{v_a^2}{2g} + p_a H_n$$

$$h_a + \frac{w_a^2}{2g} = h_l + \frac{w_l^2}{2g} + p_l H_n$$

$$h_l + \frac{w_l^2}{2g} = h_d + \frac{w_d^2}{2g} + p_d H_n$$

$$\frac{v_e^2 - w_e^2 - u_e^2 - v_a^2 + w_a^2 + u_a^2}{2g} = h_d - h_w$$

$$+ H_n (p_s + p_e + p_a + p_l + p_d) + \frac{w_d^2}{2g} \quad \ldots\ldots(6)$$

From Fig. 17, it follows that,

$$h_d - h_w = H_n \quad \ldots\ldots(7)$$

As already explained, H_n is the desired head at the exit from the casing. The friction head necessary to overcome the losses in the delivery pipe has not been considered, because these losses may vary between wide limits and are dependent on the length and cross-section of the pipe. The velocity head, $\frac{w_d^2}{2g}$, at discharge from the pump is a certain fraction of the desired head, H_n. This fraction can be represented by the coefficient, α, so that,

$$\frac{w_d^2}{2g} = \alpha H_n \quad \ldots\ldots(8)$$

The coefficient, α, gives the magnitude of the exit losses, and varies with the desired pressure head when the velocity, w_d, is constant.

In order to simplify Equation (6), let,

$$(p_s + p_e + p_a + p_l + p_d) = p \quad \ldots\ldots(9)$$

Substituting Equations (7), (8), and (9) in Equation (6), the general form of the fundamental equation is obtained,

$$v_e^2 - w_e^2 - u_e^2 - v_a^2 + w_a^2 + u_a^2 = 2g H_n (1 + p + \alpha) \quad \ldots\ldots(10)$$

Let,

$$(1 + p + \alpha) = \eta \quad \ldots\ldots(11)$$

In order to obtain the total head, H_b, which is used in the calculation of centrifugal pumps and represents the theoretical head which the pump can deliver, considering no losses whatever, the desired head, H_n, can be multiplied by the factor, η. Hence,

$$\eta H_n = H_b \quad \ldots\ldots (12)$$

From the geometric relations:

$$v_e^2 = u_e^2 + w_e^2 - 2 u_e w_e \cos \delta_e \quad \ldots\ldots (13)$$

and

$$v_a^2 = u_a^2 + w_a^2 - 2 u_a w_a \cos \delta_a \quad \ldots\ldots (14)$$

Substituting in Equation (10) the values of v_e^2, v_a^2, and $(1 + p + \alpha)$ from Equations (11), (12), and (13), the second form of the fundamental equation is derived,

$$u_a w_a \cos \delta_a - u_e w_e \cos \delta_e = \eta g H_n \quad \ldots\ldots (15)$$

DISCUSSION

J. W. MacMeeken,* Esq.—The theory expressed by the author is interesting in view of the fact that other recent research and experimental work has proved the reliability of accepted theories of the centrifugal pump. This novel explanation of the performance of these pumps is possibly applicable to machines designed to operate in a particular way, but it could not be applied to many centrifugal pumps of very recent design giving higher efficiencies than were obtained a few years ago and which show, under test and analysis, a greater generation of head for peripheral speed than can be explained by the author's theory. Accurate measurements of output and input energy and measurements by laboratory methods, of each of the mechanical losses, including disk friction, show that in a pump which is designed throughout for very high hydraulic efficiency, the head obtained can only be explained by the full effect of Equation (*B*), no element being omitted. Equation (*C*), on the other hand, is not strictly a practical equation because it assumes suction conditions that seldom exist. The author states that the fundamental laws of centrifugal pumps seem never to have been properly explained and that many misconceptions exist, even among those supposedly familiar with the hydraulics of centrifugal pumps. He further refers to fallacies and proclaims, referring to Equations (*B*) and (*C*), that "they do not, and cannot, furnish a rational basis for the analysis and design of centrifugal pumps." There is very strong evidence that he is wrong in discarding mathematical facts and replacing them by rational reasoning.

The increasing commercial value of pump efficiency, when obtained by designs that are economical for manufacture, resulted in a thorough study of the problem by the speaker and his associates, in America as well as with consultants in Europe. The theory was studied in a variety of ways and Equation (*B*) has been recognized as correct. To prove the theory experimentally, special pumps were designed and built, in order to deliver as nearly as possible the theoretical head, $\frac{V_2^2}{g}$, reduced only by the unavoidable slight rotational head of the pump suction. Some of these pumps gave a head of $\frac{0.85\ V_2^2}{g}$, or $\frac{1.70\ V_2^2}{2\,g}$. One pump, built in Europe, gave a total head of $\frac{0.96\ V_2^2}{g}$ at shut-off and practically the same at designed conditions. As 96% efficiency is the highest that can be expected, if friction and shock losses are reduced to a minimum, this pump proved that the total head, $\frac{V_2^2}{g}$, was being generated according to theory. However, these pumps were costly to manufacture and were not suitable for the American competitive field. They also showed some instability due to their susceptibility to air in the water.

Equation (*B*) having been proved experimentally by pumps of special design, a large number of different types and sizes were planned to include features more suitable for economical manufacture. This equation was used

* Engr., Byron-Jackson Pump Mfg. Co., Berkeley, Calif.

for head calculation and every hydraulic consideration, known to result in high efficiency was introduced into the design. The pumps were built with unfinished interior water-ways; and, when tested, they not only delivered the capacity and head for which they were designed, but gave the same shape of characteristic to within 1%, or 2%, as the designer had expected. The results in these cases pointed to Equation (*B*), when used with knowledge of the mechanical and hydraulic losses in the pump, as giving the accuracy obtained in designing electric motors. The effect of a change in suction design could also be computed by Equation (*B*). These pumps were of the volute design of an improved type and the volutes were of such a form that some remaining kinetic energy is discharged into the discharge cone of the pump where it is partly converted into pressure. In all cases, the head obtained could only be explained by the full effect of Equation (*B*), nothing being omitted.

In the pumps referred to by the author, which have a concentric case, it is possible that this exit velocity component is not discharged but retained unobstructed in the pump-casing and, therefore, no power is used in maintaining it. This would explain satisfactorily the difference between the author's theory and Equation (*B*) and between efficient volute pumps and efficient pumps of the author's design. This could be expressed in the following equation:

$$H = \frac{1}{g}\left(C_2\, V_2 \cos\, \alpha_2 - C_1\, V_1 \cos\, \alpha_1\right) - f\,\frac{V_3^2}{2\,g}$$

in which, V_3 is the velocity in the concentric case and f is a factor equal to, or less than, unity and possibly zero, depending on the design of the exit orifice, $\frac{V_3^2}{2\,g}$ being subtracted because it is not allowed to leave the pump, as in a volute type. This equation is used in the design of multi-stage pumps with concentric cases and permits the application of accepted and proved theory to both volute pumps if f is zero, and to pumps of the author's design, if f approaches unity.

The author's design will always require a greater impeller diameter than that required in the volute pump of good design. Therefore, pumps of large size involving thousands of horse-power and high head may reach 35% approximately more weight and cost than is necessary, and higher disk stresses in the impeller. It is noticeable that the most efficient volute centrifugal pumps of certain types generate a head greater than $\frac{V_2^2}{2\,g}$ throughout the entire range of capacity.

The author fails to be convincing in certain respects. His illustration of the hammer thrower is not applicable. In a centrifugal pump there are an infinite number of balls, all without restraining wires and all acting on one another. The final energy is the integral of the infinitesimals between limits of entrance and exit. The same error is repeated in the illustration entitled "New Conceptions of Pump Action". He mentions the case of a circular disk with radial grooves in which a smooth ball travels as the disk rotates. This is correct as regards the dynamics of a particle or a single mass. Were

this illustration applicable to a pump with radial vanes the head would be $\frac{V_2^2}{2g}$ at shut-off and $\frac{V_2^2}{g}$ when flow occurs. In the centrifugal pump there is a disk full of such balls, and the total head is the sum or integral of their combined energy. Integrating between the limits of r_1 and r_2 and expressing the velocities as functions of angular rotation and radius, Equation (B) is again the result, which shows the same head, $\frac{(V_2^2)}{g}$, at shut-off and all capacities for radial vanes. The illustrations in Fig. 2 apply very nearly to a lawn sprinkler, but not to a centrifugal pump. It has been proved experimentally that a volute or guide-vane, restraining the water from flying out of the impeller, does not result in appreciable losses. Pumps so constructed show very high hydraulic efficiencies and generate high heads, much higher than $\frac{V_2^2}{2g}$.

There is an apparent discrepancy in referring to Fig. 4 (b). The increment of power is equal to the capacity multiplied by the head. This means that the efficiency at small capacities is 100%, which is impossible if the hydraulic efficiency is considered. Certain losses are proportional to $\frac{V_2^2}{2g}(1 - k)$, in which, k is the ratio of the actual capacity considered to the designed capacity. These losses, therefore, will be great at very low capacities.

Referring to the objection to guide-vanes: These are not necessarily used to reduce the velocity, C_2. They have a useful function like that of a volute in controlling the output from the impeller. They prevent the water from flying out from the impeller and can do so with little reduction in efficiency; and they assist in obtaining high peripheral speeds with low impeller velocities, in cases where a volute presents structural difficulties.

Referring to the author's explanation of the performance of a volute pump (Fig. 8), an entirely different explanation could be given. Equation (B) may be transposed into the form:

$$H = \frac{V_2^2}{2g} + \frac{C_2^2}{2g} - \frac{W_2^2}{2g} - \frac{V_1^2}{2g} - \frac{C_1^2}{2g} + \frac{W_1^2}{2g}$$

In the event that,

$$\frac{C_2^2}{2g} - \frac{C_1^2}{2g} = \frac{W_2^2}{2g} - \frac{W_1^2}{2g}$$

then,

$$H = \frac{V_2^2 - V_1^2}{2g}$$

which would explain the performance. The method used by the author is incomplete in that it takes no account of the ratio of Dimension A to Dimension C. According to his method only the product of Dimensions A and C is effective. It is known that a change in the ratio of Dimension A to Dimension C will affect the characteristic curve very greatly.

The circular-case pump, referred to by the author has an efficiency of 79 per cent. The commercial efficiency of present-day pumps designed for this condition is 77%, not 60%, as stated by the author. Its efficiency could be raised to 79% by exact workmanship in the foundry and extra good mechanical condition. A good commercial pump of this type does not show falling off of the efficiency with increase of speed as shown in Fig. 12. When there is an obstruction in the volute, the efficiency has been found to fall with increased speed, as shown in Fig. 12.

The pump in Fig. 14 shows an efficiency of 78 per cent. A commercial pump, of the same specific speed type and about the same size, is rated at 80 per cent.

In the case of multi-stage pumps the circular case has been used by many manufacturers since about 1911 at least. Impellers of the same design have been found to give very much greater efficiencies in multi-stage volutes. This indicates again that the volute is superior.

Much misunderstanding has arisen regarding calculations for centrifugal pumps because old designs were inefficient compared with later machines in which 88.5% and even higher efficiencies have been obtained on very accurate tests.

Undefined losses led designers into empirical design constants that had no dynamic basis for their value. It is only when high efficiencies are obtained, that the losses can be analyzed with accuracy and the fundamental equation used in its theoretical form.

A centrifugal pump developing less head than that expressed by Equation (*B*) is a machine that is leaving uncompleted work and represents overexpenditure of material for work done. This is of great importance in very large pumps and might even warrant the restriction of impeller diameter, by contract specifications.

Large pumps, like large hydraulic turbines, are generally designed for as high a rotative speed as possible, with only a small margin of safety below the speed at which cavitation would occur. It has been proved that the design of the volute is important in avoiding cavitation. At very high speeds and heads a circular case with an orifice outlet is known to be noisy in large machines.

Equation (*B*) has a very wide field of application and is used by designers in hydraulic turbines, hydraulic meters, fans, and blowers. In each application designers have used their ingenuity to obtain the best results. The speaker has found it not only a useful and reliable means of designing centrifugal pumps, but also a means of analyzing results and obtaining indications of possible improvements.

THOMSON KINGSFORD,* ESQ. (by letter).—The author deserves great credit for the way he has handled his subject, and for the results he has obtained. The writer has had occasion to check several pumps against the principles given in the paper, and finds that the tests agree substantially with the results given in the author's experiments. Unquestionably the efficiency found

* Partner, Kingsford Foundry and Machine Works, Oswego, N. Y.

in pumps designed according to these principles as recorded in the paper is high, considering the pump size and other conditions of operation.

The author's Fig. 16 is particularly interesting as a comparison between two similar pumps, one of the old type with volute casing, and the other with the circular casing, as indicating the increased head obtained by the circular design. He does not state the differences in efficiency, but presumably this is in favor of the circular casing; at least it should be. Of course, the increased head is due to the specially designed impeller and larger casing. A comparison between the corresponding efficiencies would be of interest.

SANFORD A. MOSS,* ESQ. (by letter).—It is true, of course, that there are many misunderstandings in connection with the mathematics of centrifugal pumps. However, the fundamental equations, as usually given, are correct. If they are used by any one who really understands what they mean, they lead to correct results. The author mentions a new analysis, but he gives no detailed equations. It is quite probable that if these detailed equations were included, they could be shown to be exactly the same as those in current texts, properly interpreted, so that there would really be nothing new.

One item must be clearly borne in mind, that is, that a set of equations, or a set of computations, written for flow through a centrifugal pump, applies to a condition with a single load or quantity of flow. The vanes at the impeller inlet and the diffuser inlet (or volute angles if there are no diffuser vanes), must suit this one condition. This condition is a single point on the pump characteristic curve, presumably at the point of maximum efficiency. Other points on the curve—including conditions when the pump is shut off or running without flow, and when discharge is wide open—are those that appear when the various vane angles are incorrect. Hence, analysis for these conditions cannot be made on the same basis as that where maximum efficiency is expected.

Another item that must be borne in mind is that the set of equations for the occurrences in the pump apply primarily to the pump impeller extending from the impeller vane inlet to the impeller vane exit. At the impeller vane exit is a circular entrance to a region beyond the vanes, which may be called the diffuser or volute. At the entrance to this region and at the load or flow for which computations are made, there is definite pressure due to centrifugal force acting in the impeller, and a definite absolute velocity. Each of these effects represents energy put into the fluid by the impeller. The existence of a portion of fluid under the pressure found at the impeller exit means that a certain amount of energy was exerted to get the fluid into this region. The absolute velocity of the fluid similarly represents a given amount of kinetic energy. The fluid at the point in question certainly has the two additive amounts of energy in question. If the author means to point out that it is a fallacy to so consider the matter, his conclusion is incorrect.

The author gives frequent mention of a new design of pump which he has built and which shows signs of promising results. However, no details are

* Thomson Research Laboratory, General Electric Co., West Lynn, Mass.

given showing the nature of this design. The results obtained are so interesting that a desire is created for a description of details. Readers of a technical paper always like to have all the facts that are necessary to enable them to make use of the information given by the author.

P. L. REED,* M. AM. SOC. C. E. (by letter).—The author has probably noted that centrifugal pumps under test do not produce nearly as great a head as that computed by rational formulas based on the generally accepted theory, which is, as he states, that for very small rates of flow the water, as it leaves the periphery of the impeller, has a velocity nearly equal to that of the tips of the vanes and, hence, a velocity head of $\frac{U\,a^2}{2\,g}$; and that it has, in addition, a pressure head of the same amount, or a total head of $\frac{U\,a^2}{g}$. With considerable rates of flow the total theoretical head is modified by the shape of the vanes, but there is still a marked difference between the theoretical head and the actual head.

He meets this condition by boldly asserting that the fundamental theory, which is supported by Rankine, Church, and many other authorities, is all wrong and that for small discharges the water, as it leaves the impeller, has a velocity head about equal to $\frac{U\,a^2}{2\,g}$, or a pressure head of the same amount, but not both; and that the true theoretical maximum total head is, therefore, only one-half that computed according to the accepted theory.

The accepted theory is too well grounded in the laws of mechanics to be thus overthrown and the author's position will not be sustained. With radial vanes, the prevailing theory considers that the water just inside the tips of the impeller vanes has a velocity approximately equal to that of the periphery of this impeller, which can be diverted into a volute or guide-vanes and converted in large part into pressure. If conversion were perfect, the pressure thus converted from velocity would be $\frac{V^2}{2\,g}$, or $\frac{U\,a^2}{2\,g}$.

The author does not doubt that there is pressure inside the impeller. This pressure will not (disregarding friction) be lost during the subsequent diversion of the water into the volute or guide-vanes and the partial conversion of its velocity head into pressure head. There is undoubtedly both a velocity head and a pressure head at all points in the impeller. Both these heads increase as the distance from the shaft increases; caused to be sure, simultaneously and by the same medium, the rotation of the impeller.

A number of the author's original mathematical derivations and conclusions will be found to be unsound. One is the "Derivation by Principle of Angular Momentum", where the force, which is undoubtedly distributed over the length of the vanes of the impeller, is "conveniently regarded as concen-

* Capt., C. E. C., U. S. N., Bureau of Yards and Docks, Navy Dept., Washington, D. C.

trated" at their tips. This error exactly offsets or compensates for ignoring all centrifugal effects in this derivation, and the correct formula results.

The difference in the action between the round ball in the disk and the water in the impeller is that in the former case the centrifugal force is opposed only by the inertia of the ball, which receives a high radial velocity, while in the latter case the water is under restraint and is given a low radial velocity against resistance.

The author is misled by considering that the present pump theory is purely kinetic. The velocity head, represented by free radial velocity of the water produced by an impeller without a chamber, is not absent, but is represented by a pressure head in the encased impeller of a pump.

He presents his new theory of three orifices. As a rational theory it does not appeal. It implies that the head required to produce the velocity at the entrance of the impeller vanes is 100% lost in passing through the impeller; that the head required to produce the velocity at discharge from the impeller is 100% lost in passing around the volute; and that the velocity head at the throat is 100% lost in passing into the discharge pipe; or, at least, that the sum of these three hypothetical losses plus the loss at no discharge is the total loss of head in the pump. The only rational feature of this theory, which the writer can appreciate, is that the head losses due to discharge increase with the square of the quantity discharge. As an empiric formula it seems to have much merit when applied to the three pumps tested by the author. Whether it would be equally satisfactory if applied to pumps of different proportions would have to be developed by further experiments, which is the defect of all empiric rules.

Having pointed out what are believed to be fundamental defects in the author's theory, the fact remains that the actual head of a pump, operating with no discharge, is approximately $\frac{U\,a^2}{2\,g}$, and that this head is never much greater when there is a discharge (unless the pump has forward curved vanes), although the conventional application of the established theory would give a head nearly twice as great (for radial vanes). It will not do to say that the difference between the head, thus computed, and the actual head is represented by losses in friction, eddies, etc., as this would imply that the losses were nearly 50% of the total; whereas the actual measured efficiency of the pump in raising water often exceeds 70 per cent. With these facts at hand, attention is invited, not to the established theory of vortices, of which the centrifugal pump is a modified example, but to the conventional application of this theory to centrifugal pumps.

It has been noted that the actual head developed is considerably less than the rationally computed head and that all this difference cannot be explained as loss since it is not reflected in the efficiency of the pump. It must then be explained by some incorrect assumption in the application of the fundamental theory to the actual operation of the pump. The conventional application assumes that the water leaves the impeller in a direction, with respect to the impeller, parallel to the tips of the vanes. Now, if the water in the

outer part of the impeller lags behind, if it spills to some extent over the following impeller vane, the effect would be similar to bending back the impeller tips, thereby reducing the theoretical total head of the pump, and reducing, if not removing, the discrepancy between conventionally computed and actual heads. This seems plausible for other reasons. The water is being constantly pressed and accelerated tangentially by the vanes as it passes through the impeller; and near the periphery it may escape this pressure in part by lagging behind and passing through the periphery at a sharper angle than the backward curved tip of the impeller vane and at a lower absolute velocity than is conventionally assumed. This would be particularly the case with a small discharge, which would accord with test results. This effect would be increased by any cohesion or viscosity that the liquid may possess, since the tangential velocity of the liquid just outside the impeller is no doubt considerably less than it is inside.

MAX M. MUNK,* ESQ. (by letter).—The writer would like to register his disagreement with the author's conclusions, believing the old established blower theory to be correct.

In declaring the pressure to be "part and parcel of the velocity", Professor Sherzer flatly denies to the fluid pressure an existence of its own. This is in contradiction to the most fundamental conceptions. Each fluid particle is under the definite action of the adjacent particles, regardless of its motion together with them. The intensity of this action is called the pressure. How definite it is is demonstrated by the fact that the density of the fluid is influenced by the pressure. Velocity and pressure do exist independently of each other. Even the untrained man understands that perfectly well if he has ever been in a dense crowd. He moved with the crowd at a certain speed, and independent therefrom was pressed more or less by his neighbors.

The pressure created by the centrifugal force is not even a function of the exit velocity, U, only. The rotating fluid does not necessarily extend to the center, and the pressure created depends on the width of the rotating fluid ring. If the entrance radius is almost as large as the exit radius, the fluid hardly receives any increase of pressure, although it may be impelled to the same exit velocity, U, as a fluid entering at the center. The rotation of this narrow fluid ring, rather than the case of a rotating fluid disk, is illustrated by the whirling hammer.

As a matter of fact, good centrifugal pumps may furnish a pressure surpassing one-half the theoretical maximum pressure, that is, surpassing the pressure claimed as maximum by Professor Sherzer.†

ELMO G. HARRIS,‡ M. AM. SOC. C. E. (by letter).—It is evidence of a certain kind of courage on the part of the author to announce that all the heretofore recognized present-day authorities on centrifugal pumps are entirely wrong. That is a broad statement. The writer cannot concede that the author has established any one of the three claims made in the Synopsis.

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† Compare, for instance, "Die Kreiselpumpen," by C. Pfleiderer, Berlin, 1924, p. 97.

‡ Prof. of Civ. Eng., Missouri School of Mines, Rolla, Mo.

They are so sweeping that to disprove all of them systematically would require a treatise on the subject. It would be inappropriate to fill these pages with matter that can be found in many recent publications. However, the following comments may be appropriate.

(*a*).—The quotation from John Perry and the author's statements will apply to many of the writings that are more than twenty-five years old, but not to the more recent ones.

(*b*).—The author is right in declaring that Bernoulli's theorem should not be applied in the case of water flowing through a centrifugal pump, for energy (head) is being continuously added to the water as it passes through the pump. Moreover, the pressure is not constant over any cross-section of a passage between propellers. It is relatively high next to the front side of a propeller and low on the back side. Doubtless, in some defective designs, a vacuum occurs at the back side of the runners. Also, in attempting to apply Bernoulli's theorem, some writers have used relative and absolute velocities indiscriminately to the utter confusion of the subject.

(*c*).—Under "Current Fallacies", the author's analogy of the "hammer" is incomplete. He should have considered a smooth rod with hammers strung on it from the center of rotation out to the last one.

(*d*).—The writer thinks that the author is entirely wrong in his statement that the head in the water at "shut-off" is $\frac{U_a^2}{2g}$ instead of $\frac{U_2^a}{g}$. This pressure head at shut-off has been the cause of much confusion, especially among the earlier students. It led one writer to state (in a book on centrifugal pumps) that the curve of the vanes was of no consequence. The confusion results in studying, and deriving conclusions from experiments on, pumps that are not in operation—not discharging, although revolving.

The author states that "there is a pressure created by centrifugal force which is equal to $\frac{U_a^2}{2g}$ and which is part and parcel of the velocity itself. The total head may be regarded as either, but not both". This statement does not make anything clear nor remove confusion. By the laws of mechanics the head, $\frac{U_a^2}{g}$, is there, although it cannot be utilized until the pump is discharging.

(*e*).—The "Derivation by Principle of Angular Momentum" is correct, although given by the author as incorrect. Equation (*C*) can be easily transformed into the writer's Equation (37),* discussed later. However, the writer prefers to use plain reaction of a jet to the change in angular momentum. The latter term does not look familiar to ordinary students of hydraulics.

(*f*).—Under "Reasons for Observed Discrepancies", no adequate reasons are given.

(*g*).—As bearing on most of the matter relative to "Real Action of Centrifugal Pump", in the references cited later, it is proved that with β_2 fixed, it is theoretically immaterial whether the water moves out under purely centrifugal action producing pressure; or centrifugal action producing accelera-

* "Compressed Air," 1917.

tion; and it is immaterial as to the point between center and periphery at which the water is picked up; and that the same effect will be obtained by short vanes or cups picking the water up impulsively; and that the writer's Equation (37) previously referred to, applies either with or without back pressure at the discharge. The discovery of this fact was a surprise and a revelation to the writer, and it much simplifies the whole subject. For these reasons the general term, centrifugal pump, is a misnomer. Engineers may be more ready to accept these statements if they will consider the action between a golf club and the ball. Also, consider the theory (nearly realized) in the Pelton water-wheel. By the laws of mechanics, if the rotation of the wheel is reversed and water is fed to the cups, it will be thrown off with a velocity twice that of the cups. Also, consider reversing the rotation of an axial flow turbine.

(*h*).—Under "Experimental Verifications" the author interprets results by his own erroneous theories. He seems not to have given due recognition to the fact that for small discharges (as compared with discharge at best efficiency) most of the work put into the pump is dissipated. Moreover, the observations under experiment cannot be properly interpreted without knowing the area of the discharge outlet, of the pump, and the angle, β_2. With this information and by the use of the writer's Equation (37) discussed subsequently, the experiments could be interpreted correctly—not otherwise.

(*i*).—Under "Effects of Casing": If, in the experiment, the pressure head, h_2, at the line of discharge (Point 2, Fig. 1) had been determined, the total head in the water at the discharge line would be $\frac{C_2^2}{2\,g} + h_2$, and if this were plotted on Fig. 6, the line would lie above the "Actual Head" line—the difference being the loss of head beyond the discharge line.

In the analogy of the paddle in a tank—whence comes the jet which, by inference, has a higher velocity than the paddles?

(*j*).—Under "Pump Considered as Three Orifices", the author is not correct in considering as orifices the three sections that he selects in Fig. 8. The sections, 2 and 3, are not orifices. He is also incorrect in computing the drop in pressure head at each so-called orifice as that which would occur if the water, at each place, started from a state of rest. His calculations show only slight differences of velocity at the three sections and what change occurs will be gradual. The three-orifice theory is groundless.

Under "Applications of New Theory": The outstanding item of interest in the paper lies in the fact that the author has designed a very efficient pump, but that is of little value unless he will give all its dimensions, angles, etc. With these data, the results of the experiment can be interpreted in the light of other theories.

The writer is unable to see that the three-orifice theory influenced the design in any way. The author states, in reference to the "Results of First Design", that "the runner, of the double-suction type cast in bronze, was of ordinary design and workmanship."

When, if at all, the dimensions and angles of these pumps are revealed, there will be shown a small angle between the backward tangent and V_a,

Fig. 1 ($180 - \beta_2$), and, when computed, V_a will be found nearly equal to U_a. This will make the absolute velocity, C_2, nearly radial and relatively small. (It is not possible to make it radial.) Thus, there will be no "drag" between the water in the circular receiving chamber and that escaping from the runners; and there will be but little kinetic energy to lose. If the author will put in a runner with as much forward pitch as it now has backward pitch (β_2 between 20° and 30°), the efficiency will drop below 50 per cent. This drop will be caused by the loss of most of the kinetic energy in the water, due to the drag against the relatively slow moving water in the outside circular receiving chamber. If the author will attach a pressure gauge at the discharge line (Point 2, Fig. 1), he will find most of the final head registered there as pressure.

This pressure will approximate $\frac{U_a^2}{2g}$ (although always less) when the vanes are turned backward as specified. However, when the rotational velocity of the water at Point 2 is computed ($= C_2 \cos \alpha_2$), it will be found to be relatively small, altogether too small to develop the pressure by centrifugal force, hence, the writer's statement that the term, centrifugal pump, is a misnomer when applied to the majority of such pumps as are now made. Fortunately, the writer's Equation (37), developed subsequently, covers every case with the qualification that it does not include dissipated energy.

In reference to guide-vanes: They should be used with caution, if used at all. They can function properly only with one combination of velocity of rotation, quantity of discharge, and angles such that the relative motion of the water will be tangent to the vanes at their tips. Since the combination of quantity and speed of rotation that will give best efficiency cannot be determined except by test after the pump is built, it may be advisable to change the angles of the guide-vanes slightly after testing the first pump built for a stock design.

Many years ago the subject of centrifugal pumps was young as compared to the present knowledge of the theory and to the present perfection of designs and excellence of performance. The writer sought, through the available literature, for something about centrifugal pumps, that he might impart the information to his class. He found the field blank so far as his search extended, except that in the accepted highest authority on civil engineering at that time—Trautwine's "Civil Engineer's Pocket-Book"—he found, in substance, the statement that the centrifugal pump was a low-efficiency machine in which the lift was practically limited to 27 ft.

Above this lift the efficiency was said to fall off rapidly. The writer, therefore, started to prepare a lecture for his class. The preparation of that lecture took up his otherwise spare hours for a year or two. The findings were published by the Society* in 1903, and, again, in somewhat better form in his textbook.† In both these publications can be found derivations of the writer's Equation (37):

$$H = \frac{U_a^2 + V_a U_a \cos \beta_1}{g}$$

* *Transactions*, Am. Soc. C. E., Vol. LI (1903), p. 166.

† "Compressed Air," 1917, Chapter VIII, "Fans".

Note that cos β_2 is minus when β_2 exceeds 90 degrees. In this, the symbols are those used by the author and shown on Fig. 1, except the symbol, H, which is discussed later. This formula is derived by three independent lines of approach. It is derived by Daugherty* and also by Slocum.† That it is based on correct assumptions as to the underlying mechanical principles seems beyond successful controversion. The writer prefers his Equation (37) to any other that he has found. The angle, β_2, is a constant for any one pump. It is under the control of the designer. The speed of rotation, U_a, is under the direct control of the operator. V_a can be computed directly when the quantity of discharge has been measured. Therefore, it is evident that the H of the writer's Equation (37) can be quite accurately computed in any well-controlled experiment. It is the control of the chief characteristics of the pump by the angle, β_2, that gives the formula value in designing. It is in the interpretation of H in the formula that the experimenter and the designer need to use caution and judgment.

Let Q be the weight of water moved per second. If no energy is dissipated between intake and discharge of the pump, the work required to drive the pump would be QH; but there are always losses due to several causes. These cannot be measured or computed separately, but the total can be determined and separated into inside and outside losses.

Let D equal the dynamometer record of rate of work put into the pump, in foot-pounds; let F equal the dissipated energy between the intake and the discharge, in foot-pounds; and H_1, the head lost between the intake and the discharge. Then,

$$F = D - QH$$

and,

$$H_1 = \frac{F}{Q}$$

Let H_2 equal the head in the water immediately after escaping from the propellers at the line of discharge, referred to still outside water; let h_2 equal the pressure head measured by a gauge at the line of discharge, plus the height of the gauge above still outside water (suction lift); and C_2, the absolute velocity at the line of discharge. (To be computed after determining V_a.) Then,

$$H_2 = \frac{C_2^2}{2g} + h_2$$

If H_3 is the pressure head in the discharge pipe measured by a gauge as near the pump as possible, but clear of the eddying effect of the discharge from the pump, plus the height of the gauge above still water; and H_4 is the head lost due to imperfections in the fixed parts of the pump, made to receive the discharge and conduct it into the plain pipe; then,

$$H_4 = H_2 - H_3$$

* "Hydraulics," 1925, p. 311, Formula (169); also, "Centrifugal Pumps," 1925, p. 62, Formula (36).

† "Elements of Hydraulics," 1917, p. 262.

So far as the writer knows, this means of locating the losses in a pump and finding their relative amounts has not been clearly shown before. It may give some light to designers who are trying to improve the efficiency of their pumps.

Other interesting, instructive, and valuable conclusions can be reached from the writer's Equation (37) as a starting point, but, since most of them appear in the papers cited, they will not be given here. However, the following historic notes will be interesting and will serve to show how the leading men in practice in this subject were floundering in ignorance twenty-five years ago.

About 1903 the writer tried to interest a pump manufacturing company (one of the largest) in some patents that he had taken out. After courteous attention and some days of consideration, the manager announced that, unknown to the public, the company had for several years conducted experiments to develop a centrifugal pump that it would be willing to advertise and sell. Having spent $300 000 in the effort, it was then about ready to put a pump on the market and it could not afford the delay and cost of starting on a new line.

About the same time and with the same object, the writer was discussing his Equation (37) with the chief engineer of another company that was making and selling centrifugal pumps. This man stated unqualifiedly that the highest head that could be attained by any centrifugal pump, regardless of the shape of the propellers, would be $\frac{V_a^2}{2g}$, and as conclusive proof he cited that his company had tested several pumps, and in every case the highest head was observed at "shut-off", and that head was, for every pump, $\frac{V_a^2}{2g}$. Any one who is able to interpret the writer's Equation (37) can find the fallacy of this argument and conclusions.

In 1894, there appeared a book entitled "Centrifugal Pumps". It was written by an American manufacturer and was probably accepted as authoritative. In this book appears, in substance, the statement that water must be held in the pump through at least one revolution in order to give it the necessary "send", and that the advantage in turning the vanes backward (β_2 greater than 90°) is that the water is given a final shove outward.

While the derivations are given in the references cited, the one that the writer prefers is given here for the convenience of the reader. It is direct, short, and clear. Referring to the author's Fig. 1, it may be assumed that the pump takes up the water from a state of rest; and, to get the basic mechanical principles involved, all dissipation of energy will be neglected. This dissipation consists of surface friction, friction of water on water (eddies), and, in some poor designs, spraying of water through a vacuum. It may be further assumed that the revolving wheel is immersed in water under pressure (this pressure may be either greater or less than that of the atmosphere).

The static pressure of the water on the outer surfaces of the wheel is everywhere perpendicular to the surface (this surface includes the peripheral area of discharge of width, b_2). Evidently, this perpendicular pressure neither aids nor retards rotation. When, and where, the water is picked up by the runners it exerts a dynamic pressure (reaction) resisting the imparting of velocity. Thus, the runner imparts energy to the water. This energy (pressure and motion) is retained by the water and more is added as it passes outward under the influence of the propellers. However complex this action is at the pick-up (Point 1, Fig. 1), and between there and the point of discharge (Point 2, Fig. 1), it is all a part of work done by the pump in preparation (one might say) for its final discharge, at Point 2, against the back pressure and with a velocity, C_2. Since static pressure in the outside water does not resist rotation, the resistance of the pump to rotation must be that due to reaction of the water escaping at the periphery.

If Q is the weight of water escaping in 1 sec., the reaction in the reverse direction of C_2 is $\frac{Q\ C_2}{g}$, and the component opposed to tangential motion is,

$$\frac{Q\ C_2}{g} \cos \alpha_2$$

but,

$$C_2 \cos \alpha_2 = U_a + V_a \cos \beta_2$$

(note that $\cos \beta_2$, as in Fig. 1 is minus).

The work done by the pump in 1 sec. is this force multiplied by the distance, U_a, through which it is overcome in 1 sec. Therefore,

$$\text{Work} = \frac{Q}{g} (U_a^2 + U_a V_a \cos \beta_2)$$

If H is the total head imparted to the fluid by the pump, then work done $= Q H$; therefore, in the writer's Equation (37),

$$H = \frac{U_a^2 + U_a V_a \cos \beta_2}{g}$$

In this formula, H is the head delivered by the pump at the periphery of the propellers (line of discharge). It is made up of suction lift, pressure head, and velocity head. It does not include head dissipated between intake to suction pipe and periphery of propellers. If the pump worked with perfect efficiency in all parts (both before and after passing the line of discharge), the work required of the motor would be $Q H$ and the lift would be H, as computed by the writer's Equation (37).

O. GRUENBERGER,* ESQ. (by letter).—The writer's criticism is intended solely for the purpose of helping to clear up any apparent discrepancies in the existing theory of centrifugal pumps.

In explanation of his new theory, the author describes how the sum of three velocity heads, corresponding to three points in the flow through the pump, can be used to calculate the actual head produced with various rates of discharge. It is difficult to understand why these three velocity heads, and only these three, should yield this result. Obviously, another point in the flow

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could be added and the result would be different. Be that as it may, the author states that the sum of the three velocity heads is to be subtracted from the "shut-off" head as found by test. If that is the procedure, no help can be gained from this method.

The author does not state whether the "shut-off" head, found by test, is identical with a calculated shut-off head, although Figs. 9, 10, and 11, indicate that such is the case. This theory also fails to explain a "rising characteristic."

In the first part of the paper, Professor Sherzer states that the shut-off head can never be more than $\frac{U_a^2}{2g}$. As a matter of actual fact it is usually larger than that. Possibly the explanation for a small increase in head could be found in the fact that the water rotates with the impeller in a radial space of the casing, with a velocity slightly in excess of that of the impeller; but this would hardly account for pressures approaching the value of $\frac{U_a^2}{g}$.

ELMER G. HOOPER,* ASSOC. M. AM. SOC. C. E. (by letter).—Consideration of the several different theories of pump design advocated and successfully used by recognized authorities in this field of engineering would lead an observer to believe that there must be more in common among the theories than is inferred in this paper. Pumps have been designed, built, and operated according to each of the different theories, and the mere fact of continued faith in any particular idea is evidence enough of a reasonable degree of success in obtaining desired results. The only satisfactory explanation for similar results from apparently different methods is diversity in interpretation of design and test factors, and in the manner of co-ordinating them.

The author's statements, that a centrifugal pump may be considered as a series of three orifices through which all the water is to flow, that each orifice requires a difference, or drop, in head equal to the issuing velocity head beyond the arbitrarily assumed locations to produce the velocity; and, finally, that the sum of these drops is to be subtracted from the static head at no flow to give the resultant head per pound of water delivered, raise questions in the writer's mind as to logical interpretation of evidence.

There can be no doubt that the formulas representing the "new theory" gave convincing results when applied to the pumps designed and tested, and that they would undoubtedly do so with many others. However, the author's explanation that difference in pressure head equal to the delivered velocity head is needed in order to create such a velocity overlooks the fact that velocity through any opening, either orifice or nozzle, is the result of the combined velocity-of-approach head and pressure-difference head. Certainly there was an appreciable velocity of approach to the so-called orifices compared with the velocity of delivery from each one. The final interpretation by the author that these "drops" in head should be deducted from the normal static head at no flow to determine the delivered head puts a different light on the problem. A simple statement that "the loss in head

* Asst. Prof., Civ. Eng., New York Univ., New York, N. Y.

through the pump is equivalent to the sum of the velocity heads mentioned", would be more effective and would give less cause for controversy. It is a substantial contribution to the common store of knowledge, however, if this seeming fact can be more thoroughly analyzed and more amply verified. The writer does not find sufficient data in the author's paper to analyze and verify the "new theory", certainly not enough to support the contention that all present theories are wrong.

Whether or not a centrifugal pump will produce a head equal to $\frac{U^2}{g}$ or $\frac{U^2}{2g}$ per lb. of water delivered, may never be settled to the satisfaction of every partisan, but either value may be obtained under proper conditions. In a pump with a circular casing and at no discharge there is likely to be a circumferential velocity producing a velocity head closely approaching $\frac{U^2}{2g}$ and a pressure head of $\frac{U^2}{2g}$. A piezometer at the circumference would register a head equivalent to the latter value; a Pitot tube inserted through the rim against the stream would register the sum of the two values, $\frac{U^2}{g}$. How much of this last value is available for delivery depends on the losses necessarily following transformation of this high circumferential velocity into the velocity in the system. For radial tip vanes, carefully designed and built, and with pump outlet designed to convert volute velocity gradually into discharge pipe velocity, a lift greater than $\frac{U^2}{2g}$ can reasonably be expected.

In any case the energy imparted to the water to lift it and to overcome all friction incident to the delivery is from work done by the impeller in its rotation. The energy equation would have to be for the whole system, including pump losses, pipe losses to and from the pump, sudden changes in absolute velocity wherever it occurred, bend and gate losses, the useful lift, and the work done by the pump. There may be some question as to the most accurate form in which the work done by the impeller may be expressed but, in the final analysis, the only difference is in form.

A constant speed pump can be really designed only for one fixed condition and output. When all passages are open and water is flowing smoothest, more energy per pound will be delivered for the work put in than under any other condition. To reduce the flow it is always necessary to close down on a gate in order to add friction or resistance to flow that the pump itself fails to offer. Every pound of water delivered then has to lose the additional friction head purposely introduced by the gate. The apparent efficiency of a pump at maximum is influenced very much by the design of the system, of which it is a part, and the efficiency curve by the relative importance of gate friction to the whole. The author cites an effective lift of 134.6 ft. at 200 gal. per min. and an effective lift of only 98 ft. for 800 gal. per min. For full range of operation in one setting, delivery head must be fixed at 98 ft., and for all

flows less than the maximum the gate must serve to provide, directly or indirectly, friction loss or waste energy equivalent to the difference between the 98 ft. and the effective lift corresponding to that discharge. For example, in order to reduce the flow from 800 gal. per min. to 200 gal. per min., the gate must be closed so that every pound of water passing will lose 36.6 ft. lb. of energy.

F. THEODORE MAVIS,* ASSOC. M. AM. SOC. C. E. (by letter).—The author states repeatedly that the "present" theory of the action of centrifugal pumps as presented in standard textbooks is incorrect, but he does not explain his reasons very clearly. He states that Equations (*B*) and (*C*) "form the basis of all present theories", whereas the equations follow as a consequence of the hypothetical properties of an ideal fluid and its orderly behavior as it flows through an imaginary pump. The assumptions and corollaries from which these equations follow, regardless of the mathematical tools which are used in deriving them, may be summarized somewhat as follows:

I.—The Fluid:
- (1) Homogeneous.
- (2) Incompressible.
- (3) Non-viscous.

II.—The Flow:
- (1) Vortex free (that is, there is no flow in closed paths within the pump).
- (2) No energy losses.
- (3) Relative velocity at every point on a circle concentric with the axis of rotation is the same.
- (4) Steady; the rate of discharge through the pump does not change from instant to instant.
- (5) Continuous; all pump passages are filled at all times with the fluid.

III.—The Pump:
- (1) Frictionless.
- (2) The surfaces of all fluid passages coincide with stream lines.
- (3) There are an infinite number of impeller blades.

IV.—The External Forces:
- (1) Gravitational.
- (2) Mechanical.
- (3) Non-frictional.

Water is a homogeneous fluid which can be considered incompressible, but its viscosity cannot be neglected. Inseparably linked with the viscosity are resistances opposing every motion of this otherwise ideal fluid and, in the case of higher velocities, turbulence, eddies, and the inevitable consequence, loss of useful energy. The water passages of the actual pump are not perfectly smooth and frictionless and there are not an infinite number of infinitely thin impeller blades, each exerting no pressure on the water in the impeller. Since neither the water nor the actual pump meet the requirements laid down by the elementary theory, it is quite unreasonable to expect agreement between tests of pumps and calculations made according to this theory.

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The equations of the elementary theory are very useful, however, in making first approximations. They lead to the familiar straight line characteristic passing through $H = \frac{C^2}{g}$ for $Q = 0$, which may be considered as the first approximation of the actual characteristic curve.

Professor Pfleiderer* shows that a curve, very similar to the experimental characteristic, can be estimated for the purpose of design. Subsequent corrections are necessary to allow for the following factors:

1.—The effect of a finite number of impeller blades in the actual pump.
2.—Shock losses, or losses due to sudden changes in velocity.
3.—Friction losses.
4.—Leakage and recirculation losses.

The first may be made experimentally and the others, which involve losses of useful energy, can be made only by consideration of experimental data. Correction 1 may also be made analytically (assuming an ideal fluid) and as a consequence a newer theory of pumps and turbines has been developed which differs from the elementary theory before mentioned in the following details:

1.—Circulation around each propeller blade.
2.—Between two impeller blades the velocity at no two points on a given circle concentric with the pump axis is the same.
3.—There are a finite number of impeller blades and their position is known.

The mathematical analyses of the flow through pumps according to the newer theory, which considers a finite number of impeller blades, are interesting as problems in applied mathematics, but at present they are so difficult and so restricted in their application to ideal cases that they will not be likely to appeal to the designer. From a purely mathematical standpoint, Professor Spannhake† studied the flow through a pump with from one to eight radial blades and compared the absolute discharge velocity, C, for the pump having a finite number of blades with the corresponding absolute velocity, C_∞, for the pump with an infinite number of blades as assumed in the elementary theory. He also compared the corresponding moments, M and M_∞,‡ applied at the shaft of the pump and presented curves showing the ratios, $\frac{C}{C_\infty}$ and $\frac{M}{M_\infty}$, as functions of the number of impeller blades and the ratio of the radii, $\frac{r_1}{r_2}$. Table 1 shows the results of these studies.

Kucharski§ shows that the relative flow of an ideal fluid in a rotating canal may be resolved into two components: (*a*) A flow through the canal

* See Pfleiderer, "Die Kreiselpumpen," p. 71 and 123 *ff*, J. Springer, 1924.

† Wilhelm Spannhake, "Die Leistungsaufnahme einer parallel-kräenzigen Zentrifugalpumpe mit radialen Schaufeln," Festschrift zur Hundertjahrfeier der Technischen Hochschule. Karlsruhe, 1925.

‡ $M_\infty = \frac{Q\,w}{g}\,(C_{p2u}\,r_2 - C_{p1u}\,r_1) = \frac{H_\infty}{\omega}$.

§ Kucharski, "Strömung in rotierenden Kanälen," *Zeitschrift für das Gesamte Turbinenwesen*, XIV Jahrgang, Hefte 21–22 (1917).

without circulation (hence vortex free and in agreement with the assumptions of the elementary theory); and (*b*) a circulation or rotation within the canal at a constant angular velocity which, relative to the canal, is equal, but opposed in sense, to the absolute angular velocity of the canal about the axis of rotation. The circulation can be determined from measured deformations of an elastic membrane stretched (under constant stress) across a plane section of the canal perpendicular to the axis of rotation and inflated under constant (air) pressure. The deformations should be small so that the stresses in the membrane are not materially changed as a result of the deformation. Contours on the deformed membrane correspond to the stream lines; the inflating pressure corresponds to the angular velocity of the vortex relative to the canal; and the slope of a tangent to the surface of the membrane corresponds to the velocity of flow in a direction at right angles to that tangent.

TABLE 1.—TESTS SHOWING THE EFFECT OF VARYING THE NUMBER OF IMPELLER BLADES.

$\frac{r_1}{r_2}$.*	TWO IMPELLER BLADES.		FOUR IMPELLER BLADES.		EIGHT IMPELLER BLADES.	
	$\frac{C}{C_\infty}$.†	$\frac{M}{M_\infty}$.‡	$\frac{C}{C_\infty}$.	$\frac{M}{M_\infty}$.	$\frac{C}{C_\infty}$.	$\frac{M}{M_\infty}$.
0.3	0.27	....	0.46	....	0.73	....
0.4	0.21	2.50	0.38	2.10	0.68	1.60
0.5	0.16	2.00	0.30	1.80	0.59	1.45
0.6	0.12	1.65	0.23	1.55	0.50	1.35
0.7	0.08	1.40	0.16	1.35	0.38	1.25

* r_1 = inner impeller radius, and r_2 = outer impeller radius.

† C = absolute velocity of discharge for impeller with finite number of blades; and C_∞ = absolute velocity of discharge according to the elementary theory.

‡ $\frac{M}{M_\infty}$ = ratio of corresponding moments.

This elastic membrane or soap-film analogy is well known and very helpful in its application to problems of torsion and it is equally applicable to problems of hydrodynamics. This is true from a mathematical standpoint because the differential equation that describes the deformation of the thin elastic membrane is of the same form as the one which describes the vortex motion of an ideal fluid. The differential equation in rectangular co-ordinates, x and y, that expresses the deformation, f, of a thin elastic membrane from a plane position, is:

$$\frac{\delta^2 f}{\delta x^2} + \frac{\delta^2 f}{\delta y^2} = \frac{P}{T} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (16)$$

This is the same as the differential equation for vortex motion in an ideal fluid:

$$2\omega = \frac{\delta v}{\delta x} - \frac{\delta u}{\delta y} = -\left(\frac{\delta^2 \psi}{\delta x^2} + \frac{\delta^2 \psi}{\delta y^2}\right) \quad \ldots\ldots\ldots\ldots (17)$$

in which,

ω = the angular velocity about an axis perpendicular to the plane of x-y;

u = the velocity in the direction of x;

v = the velocity in the direction of y; and,

ψ = the so-called stream function.

If the stream lines are drawn for constant differences in the values of the stream function, ψ (or by analogy equal contour intervals on the inflated membrane), the same quantity of fluid flows between each pair of stream lines. The velocity, which is tangential to the stream lines, is therefore inversely proportional to the distance between these lines.

The flow through the canal, or component, a,* of the relative flow, may be determined in a few ideal cases by the methods of classical hydrodynamics, that is, by conformal representation, using functions of complex variables.† These methods are certainly to be classed with the most difficult ones in the field of applied mathematics and are quite outside the field of design. However, the determination of the flow of an ideal fluid through any given canal can be made by an experimental process suggested by Hele-Shaw which is more generally usable and vastly simpler than purely mathematical processes. This investigator has shown that, if a fluid dye is introduced through capillary tubes into water flowing between two glass plates placed close together, stream lines are traced by the threads of dyed fluid which, for any boundary conditions, agree with those stream lines calculated by the methods of mathematical hydrodynamics.‡ If a stream-line diagram is available for a given set of boundary conditions, or for a given shape of impeller blade, it can be "twisted" into another stream-line diagram with a different set of boundary conditions, by graphical methods of conformal representation discussed by Prasil.§

Oertli's studies‖ conducted in the hydraulic laboratory at the Technische Hochschule, in Zurich, Switzerland, show pictorially how the flow of water through a pump with a finite number of impeller blades differs from the flow of an ideal fluid through a pump with an infinite number of blades, as assumed in the elementary theory. He experimented with a special radial flow pump which had plate-glass windows in the upper impeller disk so that photographs could be made of the flow of the water between the blades. The impeller disks were 400 mm. in diameter and 40 mm. apart. The impeller blades were curved backward on a radius of 196 mm. and were placed so that the forward tangents to the blade and the outer impeller circle intersected at an angle of 30 degrees. A motion picture camera was mounted on the vertical axis of the pump and the relative flow of the water through the pump was recorded

* Kucharski, "Strömung in rotierenden Kanälen," *Zeitschrift für das Gesamte Turbinenwesen*, XIV Jahrgang, Hefte 21–22 (1917).

† Lamb, "Hydrodynamics," or, Lorenz, "Technische Hydromechanik," p. 275 *ff.*

‡ Hele-Shaw, "Investigation of the Nature of Surface Resistance of Water and of Stream Line Motion under Certain Experimental Conditions," *Transactions*, Inst. of Naval Archts., 1898; and "Distribution of Pressure Due to Flow Around Submerged Surfaces," *Transactions*, Inst. of Naval Archts., 1900. Photographs of such stream lines are shown in "Die Wasserbau-laboratorien Europas," p. 240 (V. D. I. Verlag).

§ Fr. Prasil, "Technische Hydrodynamik," First Edition, p. 61 *ff.*

‖ H. Oertli, "Untersuchung der Wasserströmung durch ein rotierendes Zellen-Kreiselrad," Dissertation, Technische Hochschule, Zürich, 1923.

by photographs. Aluminum filings served to indicate the movement of the water in some of the tests and a concentrated solution of potassium permanganate introduced into the water through syringe needles indicated the stream lines in others.

In the first series of tests the entrance and discharge circles were closed with plates and one of the cells thus formed between the blades was filled with water while the adjacent cell was filled with oil. The photographs of these fluids, taken as the impeller rotates, show no appreciable eddy in the oil cell, but they do show an eddy in the water with an angular velocity, relative to the rotating disk, nearly equal to the angular velocity of the impeller. If the impeller is rotating clockwise the relative eddy motion is counterclockwise, and conversely. Before the eddy is disturbed by the effect of the viscosity of the water it bears a very close resemblance to the eddy determined by the elastic membrane method.

In the second series of tests, in which the pump was operated against a closed discharge valve, eddies were again formed which behaved much like those mentioned. The viscosity of the water soon disturbed the results as before, and violent eddies, nearly circular in shape, formed between the blades at the inlet and the discharge circles of the impeller blades. Evidently, these eddies transform a large amount of the energy input into heat, and Oertli calls attention to the fact that the pump becomes warm when it is operated against a closed discharge valve.

In the third series of tests the pump was operated at three different speeds, but discharged the same quantity of water at each speed: (1) At "normal speed" which was that speed at which the water entered the impeller relatively tangential to the blades at entrance; (2) at one-half the normal speed; and (3) at twice the normal speed. The stream lines were indicated in each test by means of concentrated potassium permanganate solution introduced through syringe needles. Oertli concludes from these tests: (1) That the relative flow through the impeller is not steady, but the pulsations which occur do not seem to have any definite period. They are caused primarily by friction and consequent eddies, etc.; (2) that for two conditions of normal flow, that is, for two equal ratios of $\frac{Q}{n}$, the directions of the middle stream lines are the same. The flow at the discharge end of the impeller is less quiet for a smaller discharge than for a larger discharge since larger eddies are formed at small flows; (3) that the absolute direction of flow at the entrance to the impeller is radial regardless of the value of $\frac{Q}{n}$; and (4) that the relative velocity at the discharge end of the blades is deflected from tangential to the blades (as indicated by the elementary theory) clockwise if the impeller rotates in a counterclockwise direction, and conversely. The angle of deflection increases as $\frac{Q}{n}$ decreases.

The results of Professor Spannhake's work are in agreement with Oertli's Conclusion (4). This is evident if one constructs the conventional discharge velocity diagrams for different quantities of flow through a given pump and

reduces the absolute velocity vector in the ratio, $\frac{C}{C_\infty}$ (see Table 1), for the given quantity of water flowing through the pump.

The problems of hydrodynamics are of such a complex nature that even a study of an ideal fluid entails mathematical difficulties which have not been surmounted. The problems are infinitely more complicated if the viscosity of the fluid is taken into consideration in the primary analysis, and it is not to be expected that any theoretical equation, similar to the author's Equations (B) and (C), can be developed for the flow of water through a pump. The elementary theory as a first approximation of the solution of a given problem is by far the most logical and satisfactory one that has been proposed. The newer theory, which considers the effect of a finite number of impeller blades, indicates corrections to the velocity diagrams of the elementary theory which bring them more nearly into agreement with those determined from actual performance tests. Subsequent corrections for friction, sudden changes in velocity, and recirculation can be made only on the basis of experimental work. The characteristic curves that follow from the elementary theory and the corrections that are logically necessary if a real fluid is considered, are not unlike those determined from tests of centrifugal pumps.*

MARIO MEDICI,† ESQ. (by letter).—The author's statement as to the fallacies of modern theories of centrifugal pumps is well founded only as applied to theories that are typical of British and American texts, or of the old German method (Neumann, for instance). On the Continent, such methods, all of which are based on the Eulerian theory, are being displaced by new ideas developed from the deductions of modern hydrodynamics. This is especially true of calculations for the design of centrifugal pumps, turbo-blowers, and compressors.

Professor A. Stodola‡ and the writer§ both give (see Fig. 19) the theoretical value of H as $\frac{v_2^2}{2g}$, or $\frac{u_2^2}{2g}$, when the discharge is zero. This seems reasonable when considered in the light of the more exact characteristic equation developed by the writer for the theoretical potentiality of centrifugal impellers. However, if the discharge valve is opened a little, that is, if the discharge is almost zero, there must obviously be a certain difference between the actual and theoretical head of the impeller and the pump. This difference is the only thing that can explain the actual quantity-head curves, such as A (Fig. 19). How does the author make his theory harmonize with characteristic curves of this kind?

The writer does not concur in the opinion of Professor Sherzer as to the effect of the vane angle, β_2, on the shape of the pump characteristic. For an impeller without casing, the author accepts, as being substantially correct, the results usually given in textbooks. The writer has shown§ that at supple-

* Pfleiderer, "Die Kreiselpumpen," pp. 71 and 123 *ff*, J. Springer, 1924.

† Prof. of Mech. Eng., Polytechnic School of Naples, Italy.

‡ "Steam Turbines," VI Edition, p. 1045, by A. Stodola, Springer, Berlin, Germany.

§ "Contributo alla teoria delle turbomacchine centrifughe," by Mario Medici, *L'Industria*, September, 1926.

mentary angles of β_2 ($\beta_2 = 60°$ and $\beta_2 = 120°$, for example), the same trial characteristics occur and that the best values of β_2, for centrifugal impellers, are those between 24 and 30 degrees. This is in perfect harmony with current practice. Tests have verified the conclusion that both the vane angles, β_1 and β_2, and the number of impeller vanes, z, control the shape of the quantity-head curve.

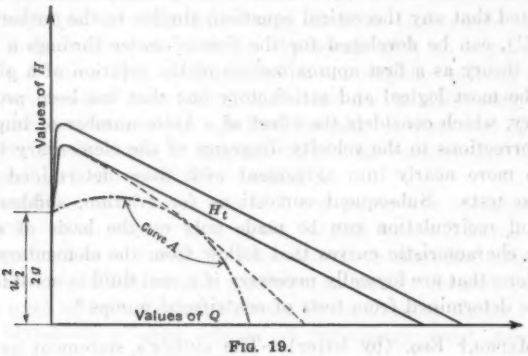

FIG. 19.

On the other hand, Professor Sherzer is to be congratulated on his conclusions as to the effect of casings and on his valuable suggestion to mount the impeller in a plain circular casing with constant cross-section. Working independently, and analyzing critically the real effect of the casing on the performance of centrifugal pumps, the writer reached the same conclusion but only with regard to series pumps (1927). The results have been published in the Italian technical press* after a delay due to a desire to obtain some proof of an experimental nature. The writer has not yet completed this experimental verification and, therefore, he is especially glad to learn of Professor Sherzer's test results as they apply to circular-case centrifugal pumps. Some of these results, as, for instance, the extreme flatness of the efficiency curves, are in perfect harmony with the writer's theoretical deductions.

However, Fig. 16 is not clear. It seems to contradict the theory given by the author. According to his theory, the head at zero discharge must be, $H_2 = \frac{v_2^2}{2g}$, or $\frac{v_3^2}{2g}$, if the effect of the side-walls is ignored. Referring to Fig. 16, if the impellers are both of the same diameter and rotate at the same speed, how can the value of the head at zero discharge in a circular casing be so different from that in a volute casing? What does Professor Sherzer mean to illustrate with the values, 284 ft. and 342 ft., as the total head at zero discharge, using volute and circular casing, respectively?

The writer hopes that these comments will not be considered as destructive criticism, but as an effort to become better acquainted with the results of this

* *L'Industria*, December, 1927.

very interesting investigation. In further support of these views, the writer submits an abridgment of his paper, "The Theoretical Potentiality of Centrifugal Impellers",* translated from the Italian and paraphrased as follows.

Designers of centrifugal pumps, fans, turbo-blowers, and compressors are recognizing, more and more, the need of abandoning the ancient Eulerian theory in favor of a more complete one. Calculations made by the old theory not only give results that are always greater than the actual values, but also (especially within forward and radial vanes) results that are far from being explained by the customary shape and size of the actual characteristic curves and the values of the impeller efficiency.

The Eulerian theory takes no account of the number of impeller vanes, z, in determining theoretical potentiality. It is based on conditions that obtain at the middle of the fluid stream passing through the impeller and diffuser vanes. In this theory the fluid trajectories and the vanes are considered congruent. With such an assumption it is easy to compute the theoretical change of energy in the fluid through the runner. This computed value will never be the same as that found under actual working conditions because the number of vanes, z, is finite, and the Eulerian theory tacitly assumes that z is infinite. In practice, it is only very close to the impeller vane that the fluid really travels in a direction that can be controlled by the inclination of the vanes. In the space between two adjacent vanes, variations both as to velocity and direction of flow appear in the stream (Fig. 20). Both these influences are responsible for reducing the actual head to less than the theoretical result as determined by considering the number of vanes infinite.

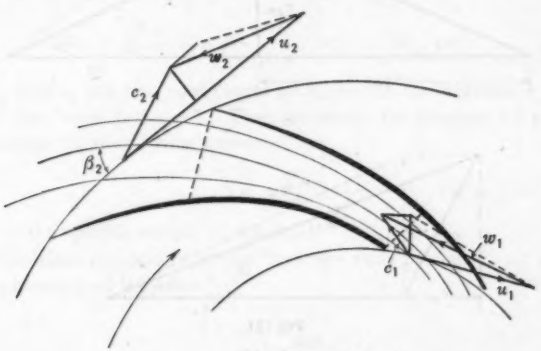

FIG. 20.

It is the variation in velocity (see Fig. 20) between vanes that forms a basis for the conception of "vane pressure"; that is, difference in pressure must undoubtedly take place between two centrifugal impeller vanes in order that power be transmitted. The effect of variations in the direction of flow is to make necessary an exaggeration of the vane angles as would be determined by the Eulerian theory.

* "Contributo alla teoria delle turbomacchine centrifughe," by Mario Medici, *L'Industria*, September, 1926.

A precise determination of the effect produced by each of the principal vane parameters (z, the number of vanes; β_{1c}, the vane angle at the inlet; and β_{2c}, the vane angle at the outlet), on the theoretical characteristics of the impeller is possible only by studying the fluid stream along the impeller; that is, by applying the methods of modern hydrodynamics, which in some respects has been as slow in making progress as aerodynamics. Under the present state of applied hydrodynamics, it seems to be much more rational to elaborate the ancient theory again, making proper allowances for β_{1c}, β_{2c}, and z, on the theoretical potentiality of centrifugal runners.

Usually, the theoretical potentiality of centrifugal impellers is expressed as:

$$H_t = \frac{u_2\, c_{2u}\, u_1\, c_{1u}}{g} \qquad (18)$$

in which,

H_t = theoretical head at the impeller;
g = acceleration due to gravity;
u_2 = peripheral speed of the impeller at the outlet;
u_1 = peripheral speed of the impeller at the inlet;
c_{2u} = the peripheral component of the absolute outlet velocity of the fluid, reduced because of "vane pressure"; and
c_{1u} = the peripheral component of the absolute inlet velocity of the fluid, reduced because of "vane pressure".

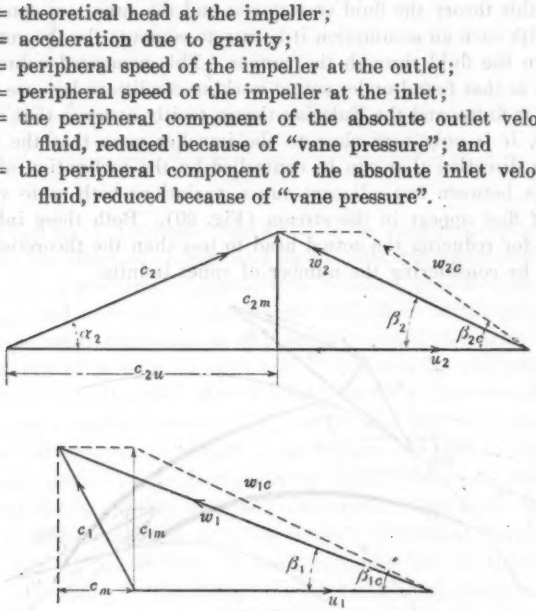

FIG. 21.

From the speed triangles (Fig. 21):

$$c_{2u} = u_2 - \sqrt{W_2^2 - c_{2m}^2} = u_2 - \sqrt{W_2^2 - \frac{Q^2}{\pi^2 D_2^2 b_2^2}} \qquad (19)$$

$$c_{1u} = u_1 - \sqrt{W_1^2 - c_{1m}^2} = u_1 - \sqrt{W_1^2 - \frac{Q^2}{\pi^2 D_1^2 b_1^2}} \qquad (20)$$

so that,

$$H_t = \frac{u_2^2 - u_1^2}{g} - \frac{u_2}{g}\sqrt{W_2^2 - \frac{Q^2}{\pi^2 D_2^2 b_2^2}} + \frac{u_1}{g}\sqrt{W_1^2 - \frac{Q}{\pi^2 D_1^2 b_1^2}} \qquad (21)$$

in which,

Q = the delivery or capacity of the impeller;
D_2 = the outlet diameter of the impeller;
b_2 = the outlet width of the impeller;
D_1 = the inlet diameter of the impeller; and
b_1 = the inlet width of the impeller.

On every cylindrical element between D_2 and D_1:

$$d\,p = \frac{1}{2\,g}\,(W_f^2 - W_b^2) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (22)$$

in which, $d\,p$ equals the difference in pressure between the two sides of a vane, and W_f and W_b are the relative fluid velocities on the front and on the back of the vane on the aforementioned element of the impeller.

According to the hypothesis of Kucharski* the relative fluid velocities on the front and on the back of the vane are quite different, but along the arcs, $A\ C$ and $E\ F$ (Fig. 22), they may be considered as uniform and equal, respectively, to W_1 and W_2. At the point, D, on the back of the vane the relative fluid velocity caused by "vane pressure" will be W_{1b} which is greater than W_1. Similarly, at Point G_f for the same reason, the relative fluid velocity will be W_{2f} which is less than W_2. At Point D_f, on the front side of the vane, the relative fluid velocity will be W_{1c} which is less than W_1, and at Point G_b, the relative fluid velocity will be W_{2b} which is greater than W_{2c} (Fig. 22). From Equation (22), then:

$$d\,p_G = \tau_2\,d\,p = \frac{1}{2\,g}\,(W_{2b}^2 - W_{2f}^2) = \frac{1}{2\,g}\,(W_2^2 - W_{2f}^2) \ldots\ldots (23a)$$

$$d\,p_D = \tau_1\,d\,p = \frac{1}{2\,g}\,(W_{1c}^2 - W_{1c}^2) \ldots\ldots\ldots\ldots\ldots (23b)$$

in which, τ_1 and τ_2 are two coefficients to allow for the difference in the distribution of the "vane pressure". They generally lie between 1.2 and 0.8. If p_m is the average value of "vane pressure":

$$d\,p = \frac{p_m}{\zeta} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (24)$$

in which, ζ is the specific weight of the fluid.

The peripherical component of the "average vane pressure", $p_m \sin \beta$, gives on a surface element of the vane:

$$d\,s = d\,b\,\frac{d\,r}{\sin\,\beta} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (25)$$

the turning moment:

$$d\,m = p_m \sin\,\beta\,r\,d\,s - p_m\,r\,d\,r\,d\,b \ldots\ldots\ldots\ldots\ldots (26)$$

Assuming that p_m is uniformly distributed over the central portion of the vane, $D_f\,G_f$ (Fig. 22):

$$m = p_m \int_{r_D}^{r_G} \int_0^{b_m} r\,d\,r\,d\,b = p_m\,b_m \int_{r_D}^{r_G} r\,d\,r \ldots\ldots\ldots\ldots (27)$$

* Kucharski, "Strömungen in rotierenden Kanälen," *Zeitschrift für das Gesamte Turbinenwesen*, 1917, s. 201.

Since:

$$\int_{r_D}^{r_G} r\,d\,r = \varepsilon \int_{r_1}^{r_2} r\,d\,r = \varepsilon \left(\frac{D_2^2 - D_1^2}{8}\right) \ldots\ldots\ldots\ldots (28)$$

in which, ϵ is a coefficient, defining the shape of the vane, then:

$$m = \varepsilon\, p_m b_m \frac{(D_2^2 - D_1^2)}{8} \ldots\ldots\ldots\ldots\ldots\ldots (29)$$

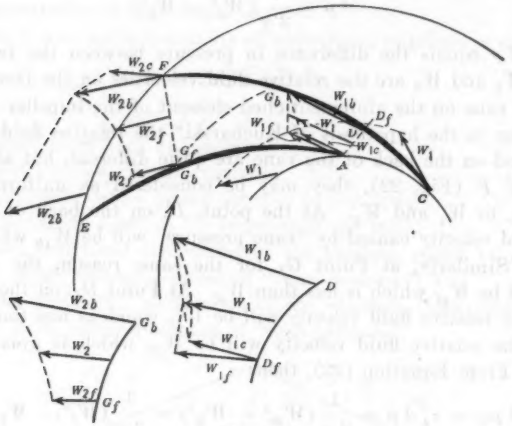

FIG. 22.

The turning moment for every vane is:

$$M = K \frac{N}{G\,z\,n} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (30)$$

in which,

K = a constant;

$\frac{N}{G}$ = the power, in horse power, required by each impeller;

G = the number of the impellers; and

n = the revolutions per minute.

therefore,

$$m = f\,M = K\,f\,\frac{N}{G\,z\,n} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (31)$$

in which, f denotes the relation between the turning moment, M_1, which corresponds to the part, G-D, of the vane, and the total turning moment, M, for every vane.

It follows that:

$$p_m = K \frac{f}{\varepsilon} \frac{N}{G\,z\,n\,b_m\,(D_2^2 - D_1^2)} = K\,\sigma \frac{N}{G\,z\,n\,b_m\,(D_2^2 - D_1^2)} \ldots (32)$$

The coefficient, σ, may vary between 0.25 and 0.6. Since:

$$N = \frac{\zeta\, Q\, H_m}{550\, \eta_t\, \eta_v\, \eta_m} = \frac{\zeta\, Q\, H_t}{550\, \eta_v\, \eta_m} \quad \ldots\ldots (33a)$$

$$\psi = K \frac{\sigma}{\eta_v\, \eta_m\, z\, n\, b_m\, (D_2^2 - D_1^2)} \quad \ldots\ldots (33b)$$

then,

$$d\, p = \psi\, Q\, H_t \quad \ldots\ldots (34)$$

Solving Equation (34) simultaneously with Equations (23*a*) and (23*b*), respectively:

$$W_2^2 - W_{2b}^2 = 2\, g\, r_2\, \psi\, Q\, H_t \quad \ldots\ldots (35a)$$

$$W_{1b}^2 - W_{1c}^2 = 2\, g\, r_1\, \psi\, Q\, H_t \quad \ldots\ldots (35b)$$

However, from Fig. 22,

$$W_2^2 - W_{2b}^2 = (W_2 + W_{2b})\,(W_2 - W_{2b}) = 2\, W_{2c}\, 2\,(W_2 - W_{2c}) \quad \ldots (36a)$$

$$W_{1b}^2 - W_{1c}^2 = (W_{1b} + W_{1c})\,(W_{1b} - W_{1c}) = 2\, W_1\, 2\,(W_1 - W_{1c}) \quad \ldots (36b)$$

so that:

$$W_{2c}\,(W_2 - W_{2c}) = \frac{g\, r_2\, \psi}{2}\, Q\, H_t \quad \ldots\ldots (37a)$$

$$W_1\,(W_1 - W_{1c}) = \frac{g\, r_1\, \psi}{2}\, Q\, H_t \quad \ldots\ldots (37b)$$

From Fig. 21:

$$W_{2c} = \frac{C_{2m}}{\sin \beta_{2c}} = \frac{Q}{\pi\, D_2\, b_2 \sin \beta_{2c}} \quad \ldots\ldots (38a)$$

$$W_{1c} = \frac{C_{1m}}{\sin \beta_{1c}} = \frac{Q}{\pi\, D_1\, b_1 \sin \beta_{1c}} \quad \ldots\ldots (38b)$$

Combining Equation (38*a*) with Equation (37*a*) and Equation (38*b*) with Equation (37*b*):

$$W_2 = \frac{g\, r_2\, \psi\, \pi\, D_2\, b_2 \sin \beta_{2c}}{2}\, H_t + \frac{Q}{\pi\, D_2\, b_2 \sin \beta_{2c}} \quad \ldots\ldots (39a)$$

$$W_1 = \frac{Q}{\pi\, D_1\, b_1 \sin \beta_{1c}} - \frac{G\, r_1\, \psi\, \pi\, D_1\, b_1 \sin \beta_{1c}}{2}\, H_t \quad \ldots\ldots (39b)$$

Let:

$$K_u = \frac{g\, r_2\, \psi\, \pi\, D_2\, b_2 \sin \beta_{2c}}{2} \quad \ldots\ldots (40a)$$

$$K_1 = \frac{g\, r_1\, \psi\, \pi\, D_1\, b_1 \sin \beta_{1c}}{2} \quad \ldots\ldots (40b)$$

then Equations (39) become:

$$W_2 = K_u\, H_t + \frac{Q}{\pi\, D_2\, b_2 \sin \beta_{2c}} \quad \ldots\ldots (41a)$$

$$W_1 = \frac{Q}{\pi\, D_1\, b_1 \sin \beta_{1c}} - K_1\, H_t \quad \ldots\ldots (41b)$$

Substituting in Equation (18), the values W_2 and W_1 as given by Equations (41):

$$H_t = \frac{u_2^2 - u_1^2}{g} - \frac{u_2}{g}\sqrt{\left(K_u H_t + \frac{Q}{\pi D_2 b_2 \sin \beta_{2c}}\right)^2 - \frac{Q^2}{\pi^2 D_2^2 b_2^2}} + \frac{u_1}{g}\sqrt{\left(\frac{Q}{\pi D_1 b_1 \sin \beta_{1c}} - K_1 H_t\right)^2 - \frac{Q^2}{\pi^2 D_1^2 b_1^2}} \quad \ldots\ldots\ldots (42)$$

Equation (42) is a fourth-power equation of H_t and Q.

When $Q = 0$:

$$H_{0t} = \frac{\frac{u_2^2 - u_1^2}{g}}{1 + K_u \frac{u_2}{g} + K_1 \frac{u_1}{g}} \quad \ldots\ldots\ldots (43)$$

but when the discharge is zero, the fundamental equation is, $\frac{u_2^2}{2g}$. Apparently, therefore, there are two values of the head when $Q = 0$; but, in reality, the value, $\frac{u_2^2}{2g}$, corresponds to the condition when $Q + Q_v = 0$, in which, Q_v denotes the portion of delivery lost in the clearances and the value, H_{0t}, given by Equation (43), applies to the useful delivery, Q. Consequently, there must be a certain skip in the theoretical head of the impeller as shown in Figs. 19 and 23.

With $H_t = 0$,

$$K_a = \frac{n}{60\, g\, b_2}\sqrt{\left(\frac{1}{\sin^2 \beta_{2c}} - 1\right)} - \frac{n}{60\, g\, b}\sqrt{\left(\frac{1}{\sin^2 \beta_{1c}} - 1\right)} \quad \ldots\ldots (44)$$

and Equation (42) becomes:

$$K_a Q - \frac{(u_2^2 - u_1^2)}{g} = 0 \quad \ldots\ldots\ldots (45)$$

and,

$$Q_0 = \frac{u_2^2 - u_1^2}{K_a g} \quad \ldots\ldots\ldots (46)$$

The graphical representation of Equation (42) is a fourth-order curve in H_t and Q. As Fig. 23 shows, this curve cuts the H_t-axis at a point, P_0, which corresponds in value to H_{0t} and it cuts the Q-axis at a point, P_1, which corresponds to the delivery value, Q_0, given by Equation (46).

This theory explains the value, $\frac{u_2^2}{2g}$, which is so often measured in actual characteristics near the point of zero discharge, and which, until now, was an unexplained feature in the old theory.

In Fig. 23 the part of the curve, P_0-P_2, that is most important is approximately a straight line. Making the assumption that it is a straight line is justified for the sake of greater simplicity and gives results that are in

accord with all the latest hydrodynamic theories. The result is the straight line equation:

$$H_t = \frac{W \Gamma z}{2 \pi g} = \frac{n \Gamma z}{g} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (47)$$

in which, Γ denotes vane rotation.

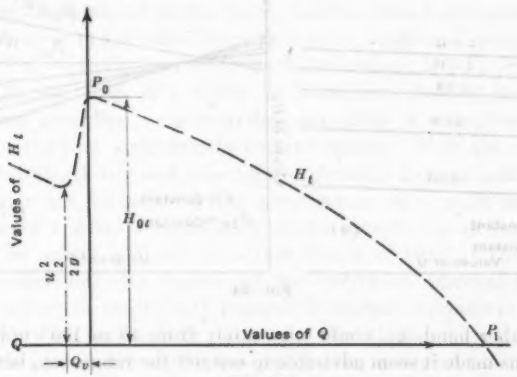

FIG. 23.

It is evident, therefore, that the number of vanes to be provided in turbo-pumps, blowers, and impellers is limited by practical conditions at two opposite extremes. Considering friction losses in the impeller ducts as the most important factor in the design, it would be desirable to use the least possible number of impeller vanes; but if the cavitation losses were considered most important, it would be necessary to avoid choosing z below a certain value, which may be easily determined by studying the performance of each specific impeller. For radial impellers, these conditions may be expressed, in metric units, as follows:*

$$z \text{ (minimum)} = 10 \frac{Q H_m}{n \eta_t b_m (D_2^2 - D_1^2)} \quad \ldots\ldots\ldots\ldots\ldots (48)$$

$$z \text{ (maximum)} = 30 D_2 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (49)$$

For an impeller with given values of D_2, D_1, b_2, b_1, β_{2c}, and β_{1c}, the theoretical characteristics, H_t and Q, can be found by assuming different values of z. Plotting these values results in a set of curves similar to Fig. 24(a) which refers to a centrifugal pump impeller. By inspection it is apparent in this case that there is little advantage gained by increasing z to more than eight or decreasing it to less than six. Increasing the number of vanes does not cause a proportionate increase in theoretical head and, conversely, decreasing z causes an ever-increasing and disproportionate reduction in head.

The proper choice of the inlet angle, β_{1c}, is also confined, normally, to a narrow range (12 to 25°), because its value is dependent, each time, on the

* "Pompe Centrifughe," by M. Medici, Capitoli III e IX, Hoepli, Milano, Italy, 1927.

best assumption of the value, C_{1m}, the radial component of the absolute speed of the water passing through the inlet. For example, Fig. 24(*b*) shows the way in which (H_t-Q)-values of a centrifugal pump impeller vary for different values of β_{1c} when z and β_{2c} are kept constant at seven vanes and 24°, respectively.

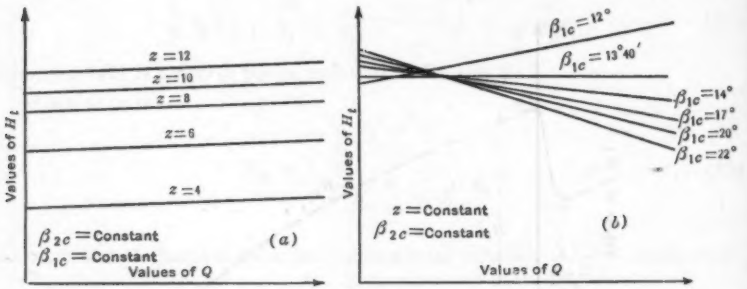

FIG. 24.

On the other hand, β_{2c} could vary safely from 10 to 170° unless practical considerations made it seem advisable to restrict the range, say, between 20 and 150 degrees. Vanes that are inclined too far backward (β_{2c} less than 20°) introduce undue friction losses and those that are inclined too far forward (β_{2c} greater than 150°) promote fluid disturbances and regurgitations within the impeller.

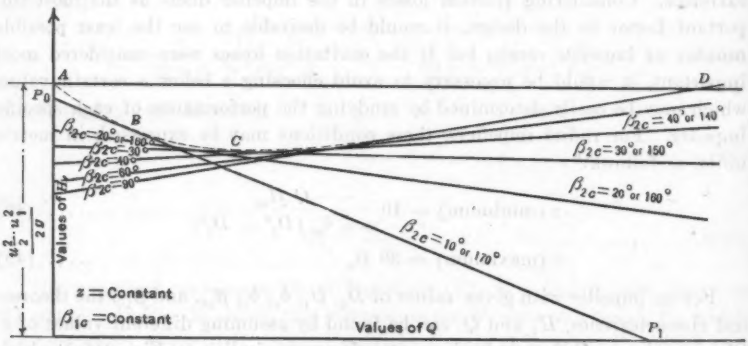

FIG. 25.

Fig. 25 is a set of (H_t-Q)-curves plotted by assuming different values for β_{2c}. Values of D_2, D_1, b_2, b_1, z, and β_{2c}, are fixed and remain constant in each case. It is apparent that, with Q equal to zero (or nearly zero), the head, H_{0t}, decreases as β_{2c} is increased from 10 to 90°; but as β_{2c} continues to increase from 90 to 170°, H_{0t} increases again at the same rate. In other words, H_{0t} has the same value for β_{2c} = 30° and 150°, for β_{2c} = 10° and 170°, etc. Furthermore, with β_{2c} fixed somewhere between 24 and 32° (depending

on the different types of impeller), the (H_t-Q)-characteristic is parallel to the Q-axis, or the head is constant, theoretically, for varying values of Q. For values outside this range, the head varies as indicated in Fig. 25. According to present theory it seems that the most effective values of β_{2c}, for radial pump impellers, lies somewhere between 24° and 30° and this is in perfect harmony with practical research findings.

The direct development of this theory leads to the establishment of a "non-effective" zone for radial impellers with a finite number of vanes. This zone is shown in Fig. 25 as the area above the dotted line, *A B C D*. Vanes inclined backward (β_{2c} less than 90°), which are considered the best for the development of head according to present-day standards of manufacture, are also shown to be the best according to present theory. With the modern, more complete analysis, theory and practice are perfectly harmonized.

The writer has investigated the performance of a multi-stage pump in order to obtain a definite verification of the present theory and, if possible, to control the values assigned to σ (see Equation (32)). These tests were made in connection with studies of the cavitation phenomenon and suction performance in centrifugal pumps.* Different depressions at the pump suction nozzle were regulated by means of a throttle flap located in the suction pipe. The head and delivery of the pump were measured after each change. The tests were repeated for three values of n: 1 450, 1 600, and 1 750.

Other values were: $D_1 = 0.36$ ft.; $D_2 = 0.82$ ft.; $b_2 = 0.023$ ft.; $b_1 = 0.0492$ ft.; $\beta_{2c} = 30°$; $\beta_{1c} = 17°$; $z = 6$; $G = 3$; P_a (atmospheric pressure) = 29.8 in.; water temperature = 64.5° Fahr.; specific weight of water = 62.25 lb. per cu. ft.; and $P_t = 0.83$ in. (water column). The suction pipe was straight and was 5 ft. in length. Packings were very carefully lined in order to introduce a minimum amount of air. The artificially produced suction heads were 23.8, 26.2, and 27.8 ft. The results of the research are shown in Fig. 26. The largest suction heads always cause the most pronounced cavitation, with a corresponding decrease in the quantity of water delivered. Values of σ were computed by use of the formula:†

$$\sigma = \frac{\zeta\, G\, z\, n\, \eta_t (D_2^2 - D_1^2)\, b_m \left[\frac{\zeta_w}{\zeta}\right\} (P_a - P_t) - \frac{C_{1a}^2 - C_1^2}{2g}\left\} - H_a\right]}{K\, Q\, H_m\, \zeta_w} \quad ..(50)$$

in which, ζ_w is the specific weight of water at — 15.56° Fahr. The results of these tests are given in Table 2.

Under the conditions of Test No. 8, which is the normal performance of the pump, $\sigma = 0.25$. Then, from Equation (40*a*),

$$K_u = 0.0856$$

From Equation (40*b*),

$$K_i = 0.0474$$

$$1 + k_u + k_i = 1.2265$$

* "Untersuchungen über das Verhalten der Saugfähigkeit der Kreiselpumpen," by M. Medici, Heft 1, *Fördertechnik und Frachtverkehr*, 1927.

† "Il fenomeno di cavitazione ed i limiti di aspirabilità delle pompe centrifughe," by M. Medici, No. 26, *Il Monitore Tecnico*, 1925.

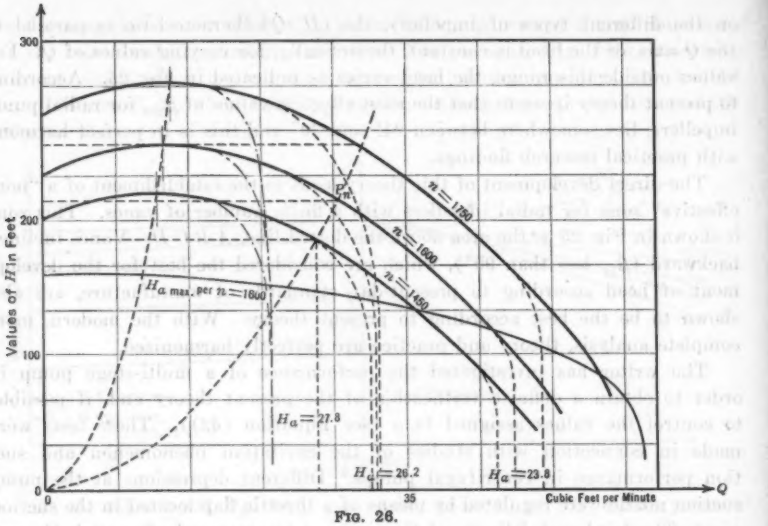

FIG. 26.

TABLE 2.—RESULTS OF TESTS ON A CENTRIFUGAL PUMP.

Test No.	H_a.	n.	Q_e, in cubic feet per minute.	σ.
1	27.8	1 450	21.8	0.364
2	27.8	1 600	21.8	0.346
3	27.8	1 750	21.8	0.318
4	26.2	1 450	31.4	0.305
5	26.2	1 600	32.0	0.3
6	26.2	1 750	32.6	0.25
7	23.8	1 450	44.5	0.27
8	23.8	1 600	46.0	0.25
9	23.8	1 750	49.0	0.24

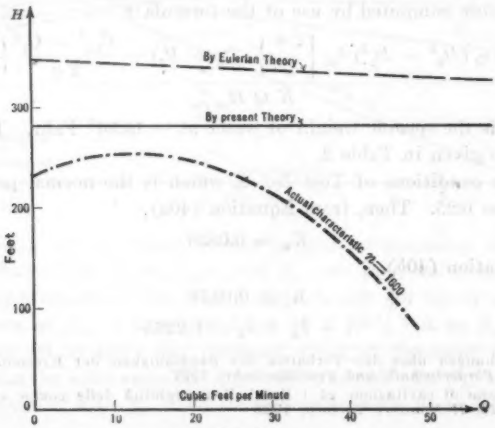

FIG. 27.

$$G\,\frac{u_2^2 - u_1^2}{2\,g} = 348 \text{ ft.}$$

$$G\,H_\alpha = \frac{348}{1.2265} = 284 \text{ ft.}$$

From Equation (44),

$$K_u = 5.75 \text{ ft.}$$

From Equation (46),

$$Q_0 = \frac{348}{5.75} = 60.5 \text{ cu. ft. per sec., or 3 630 cu. ft per min.}$$

Fig. 27 shows a comparison of theoretical characteristics (H_t-Q), obtained by the use of the new theory and the Eulerian theory and also shows the actual characteristic curve of a pump investigated when n = 1 600 rev. per min.

A. H. BLAISDELL,* ESQ. (by letter).—After a careful study of this paper, the writer cannot bring himself to the point of agreeing with the author on the basic assumption which lies at the foundation of the latter's argument. Professor Sherzer states that,

"* * * water being set in rotation in a cylindrical vessel by the action of a paddle or impeller, there is pressure created by centrifugal force which is equal to $\frac{U_a^2}{2\,g}$ and which is part and parcel of the velocity itself. The total head may be regarded as either, but not both".

The fallacy of this idea should not be difficult to comprehend.

In Fig. 28 is shown in cross-section a cylindrical casing, *a*, containing a paddle, *b*, which is attached to a vertical shaft that projects through the upper side of the casing. Assume the shaft and its paddle to be rotating with a uniform angular velocity. If the casing contains a liquid the latter will be forced into rotation and, as Professor Sherzer states, a static pressure is developed due to the centrifugal action; but, contrary to the author's viewpoint, this pressure or potential energy head can be considered, together with the velocity head, as the total energy head imparted to the liquid, if hydraulic losses are neglected.

At the outer edge of the casing are located two piezometer tubes, *c* and *d*. Tube *c* is a static tube with its lower end opening directly into the casing, and Tube *d* has its lower end projecting down into the whirling liquid, the end being turned through 90° with the opening facing directly into the flow. That is, *d* is a Pitot tube. Evidently, with the upper ends of the tubes open to the atmosphere, liquid will rise upward in both, but not to the same heights, as the author's theory indicates. In Tube *c* the liquid will rise to a height, h_a, just sufficient to balance the pressure due to the rotation of the liquid. At Tube *d*, it will be found that the liquid has mounted to a height of H ft., which is equal to twice the height of the liquid in Tube *c*. This is not strange since this tube measures the sum of the static and velocity heads. In order to

* Asst. Prof., Mech. Eng., Carnegie Inst. of Technology, Pittsburgh, Pa.

convince oneself that h_a and H can be properly considered as energy heads, it is only necessary to cut off the tops of the tubes at heights a little less than h_a and H. Liquid will start flowing out from the tops of both tubes and the discharge will continue as long as liquid is supplied to the casing. In other words, the casing and its rotating paddle is a rude sort of pump which is doing the work of lifting liquid through the heights of h_a and H.

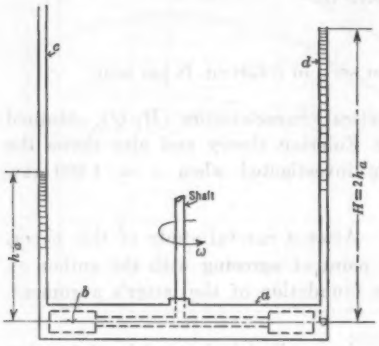

FIG. 28.

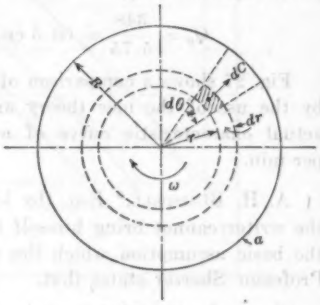

FIG. 29.

The fact that the pressure developed by the centrifugal action can also be considered as an energy head, may be demonstrated analytically as follows. Let a of Fig. 29 represent the casing of the pump in which liquid is rotating with the uniform angular velocity, ω. Consider an elementary ring of the liquid at a distance, r, from the center of rotation. In order that the shape of the ring may be maintained it is necessary that the centrifugal force developed by the rotation of the ring be counterbalanced by the pressure of the liquid against the outer surface of the ring.

The centrifugal force generated by the rotation of the section, $r\ d\theta\ dr$ (and of unit thickness), will be,

$$dC = r\ \omega^2\ dm = \frac{\gamma}{g}\ r\ d\theta\ dr\ r\ \omega^2$$

in which, γ is the density of the liquid, in pounds per cubic foot. The total pressure acting on the outer wall of this differential section can be expressed as $(r + dr)\ dp\ d\theta$, which is, $r\ d\theta\ dp$, if the product, $dp\ dr\ d\theta$, is neglected. Equating the expressions for centrifugal force and restraining pressure gives,

$$p = \frac{\gamma}{g\ \omega^2} \int_0^R r\ dr = \frac{\gamma}{g} \frac{\omega^2\ R^2}{2} = \gamma \frac{U_a^2}{2\ g}$$

in which, U_a is the linear velocity at the radius, R. Dividing the pressure by the density now provides,

$$\frac{p}{\gamma} = h_a = \frac{U_a^2}{2\ g}$$

as the pressure head (or energy) which must be added to the velocity head to obtain the total energy head at a distance, R, from the center of rotation.

With respect to the application of Bernouilli's theorem to the flow of liquids, the writer again finds it necessary to disagree with the author. Strictly speaking, this theorem, which is only a restricted form of the law of conservation of energy, applies only to flows where the motion is steady or stream like and the form of channel or nature of the motivating mechanism which maintains the flow does not come into consideration. In the case of actual liquids, the steady state can only be approximated since the effects of viscosity and boundary friction cause an eddying and turbulent motion where the kinetic energy of translational motion is only a part of that present. For these reasons it is quite necessary to apply the theorem with considerable care to engineering problems where one is dealing with real fluids.

To be sure, the theory of centrifugal pumps as discussed by Professor R. L. Daugherty* has many limitations but these have been recognized and the only excuse for the continued use of this theory has been its simplicity and familiarity as compared to the more intricate analyses obtained by use of higher mathematics. Relatively recent investigations in aerodynamics have led to a new interpretation of the classical hydrodynamics,† making it possible to analyze the fluid action in turbo-machinery with very good approximation to the actual flow phenomena. By revealing what takes place in a blower or pump, these same researches have made it possible to apply the classical theory of centrifugal pumps with satisfactory results.

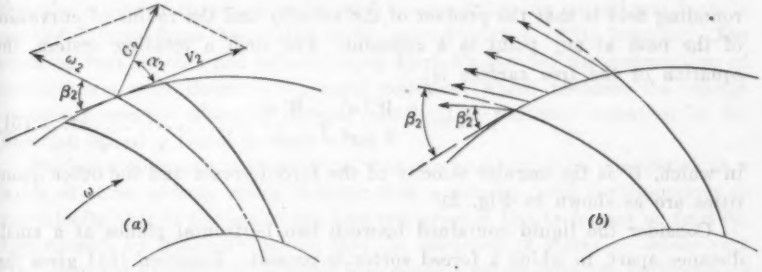

FIG. 30.

The writer believes that Equation (*B*) is quite satisfactory as it stands. Without doubt the motion of the impeller extends back into the liquid in the suction pipe giving a swirl or corkscrew motion to the approaching mass. The practical effect, in so far as Equation (*B*) is concerned, is to increase the magnitude of the term, $C_1 V_1 \cos \alpha_1$, since under this condition angular momentum is supplied to the liquid before it reaches the impeller. Aside from this direct influence of the impeller, there is (contrary to Professor Sherzer's viewpoint) energy in the liquid which enters the rotor due in most cases (if supply head is negative) to the action of atmospheric pressure. Of course, an equal amount of energy must be furnished to the liquid by the impeller.

* In "Centrifugal Pumps."

† "Hydrodynamics," by Horace Lamb, 4th Edition, 1916.

Very careful analytical and experimental work has shown conclusively that the liquid discharges from an impeller with a relative angle, β_2' (Fig. 30 (*b*)), which is less than the one indicated by the customary vector layout (Fig. 30 (*a*)). Moreover, the relative velocity of discharge at a given section of the impeller channel is not constant. For example, at the exit section, the liquid at the rear of the blades discharges with a greater relative velocity than that on the front side. Consequently, the pressure of the flowing liquid is higher in front and less on the rear, etc. However, these facts, when understood, make it possible to continue to use the classical theory.*

MORROUGH P. O'BRIEN,† JUN. AM. SOC. C. E. (by letter).—Professor Sherzer's comparison of the centrifugal pump to an athlete throwing a hammer is very apt, but not in the manner suggested. The tangential velocity determines the kinetic energy which will propel the hammer and is for this reason the only energy content of interest to the athlete; but there is also present in the rotating system an additional energy due to the stretching of the wire. In a centrifugal pump, the tension in the wire is replaced by the centripetal force exerted by the pump casing.

Probably the simplest manner of considering the shut-off condition in a centrifugal pump is to regard it as a forced vortex surrounded by a liquid moving irrotationally.‡ The condition of irrotational motion in the surrounding field is that the product of the velocity and the radius of curvature of the path at any point is a constant. For such a rotating system, the equation of the free surface is:

$$Z = -\frac{W^2 a^4}{2 g r^2} + \frac{W a^2}{g} \qquad (51)$$

in which, W is the angular velocity of the forced vortex and the other quantities are as shown in Fig. 31.

Consider the liquid contained between two horizontal planes at a small distance apart, in which a forced vortex is present. Equation (51) gives the pressure difference between the center of rotation and a point at a distance, r. It is seen that for $r = \infty$, the pressure is $\frac{W^2 a^2}{g}$ or $\frac{V^2}{g}$, and that at $r = a$, it is $\frac{W^2 a^2}{2 g}$ or $\frac{V^2}{2 g}$.

For any value of r, the vertical distance from the z-line to the value of z at $r = \infty$, represents the velocity head still present in the liquid. As the casing of a pump is somewhat larger than the impeller, but is never of infinite

* See "A Method of Analyzing the Performance Curves of Centrifugal Pumps," by J. Lichtenstein, *Transactions*, Am. Soc. Mech. Engrs., Vol. 49, November, 1927, p. 1269 (Synopsis); also, "The Hydrodynamic Theory of Turbines and Centrifugal Pumps," by Dr. Ing. Bruno Eck, *Engineering* (Lond.), January 22, 1926; and, also, "Kolben- und Turbo-Kompressoren," by P. Osterlag, Julius Springer, Berlin, Germany.

† Asst. Prof., Mech. Eng., Univ. of California, Berkeley, Calif.

‡ "The Mechanical Properties of Fluids," D. Van Nostrand, 1924.

dimensions, the pressure head at shut-off may be greater than $\frac{V^2}{2g}$, but can never be $\frac{V^2}{g}$.

When liquid is being discharged through the forced vortex and surrounding irrotational field, the determination of the pressure distribution is greatly complicated by the fact that the angular velocity of any fluid particle is not

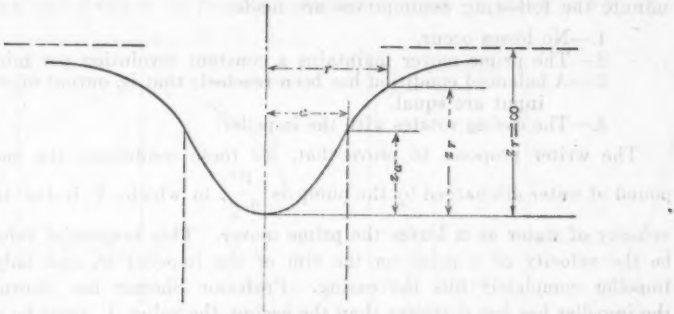

FIG. 31.

the same as that of the impeller, except for radial blades, and by the fact that the radius of curvature to be used in computing the centrifugal force is not the distance from the particle to the axis of rotation of the impeller. The equation may be obtained by combining Corioli's law for the composition of accelerations with Bernoulli's general equation, which includes the change in energy content along a stream tube.* The resultant equation is the Eulerian equation found in most texts.

Professor Sherzer's data showing that the pump may be considered as a series of three orifices, really indicate that a velocity head corresponding to the velocity that is normal to the first two areas is lost in impact or friction. Experiments by Stanton† showed that the efficiency of vortex chambers very often does not exceed 10%, a result that roughly agrees with the indication of losses shown by these experiments. As to the third orifice, at the entrance to the discharge line, the velocity head present at this point must also be present at the point at which the pressure is measured. Since there are no dimensions of the experimental pumps given, it is impossible to consider the data in the light of accepted theories. From the general description, the new design seems to correspond to some of the earliest types of centrifugal pumps. Other references to this subject may be found in theories proposed by Mr. A. H. Gibson.‡

R. T. LIDDICOAT,§ ESQ. (by letter).—The writer has studied the problem and his conclusions are squarely in line with Professor Sherzer's arguments.

* "Om Bernoulli's Ekvationer och, Ansatser Till en Komplettering av Turbinteorien," by Hjalmar O. Dahl, *Teknisk Tidskrift*, February 17, 1923, p. 13.

† Proceedings, Inst. of Mech. Engrs., 1903.

‡ "Hydraulics and Its Applications," by A. H. Gibson, D. Van Nostrand, 1925.

§ Asst. Prof., Dept. of Eng. Mechanics, Univ. of Michigan, Ann Arbor, Mich.

It seems that some of those interested in centrifugal pumps do not realize what findings Professor Sherzer has presented. If he is correct in his work, he gets an amount of energy h ft-lb. per lb. of water pumped at a given revolution per minute, whereas by the present theory he should get $2h$; but this is not all. He assumes that he pumps water without losses as do the proponents of the present theory. Both theories cannot be right. To determine the maximum energy a given pump can give to the water at a given revolution per minute the following assumptions are made:

1.—No losses occur.
2.—The prime mover maintains a constant revolution per minute.
3.—A balanced condition has been reached; that is, output of water and input are equal.
4.—The casing rotates with the impeller.

The writer proposes to prove that, for these conditions, the energy per pound of water discharged by the pump is $\frac{V^2}{2g}$, in which, V is the tangential velocity of water as it leaves the prime mover. This tangential velocity will be the velocity of a point on the rim of the impeller if, and only if, the impeller completely fills the casing. Professor Sherzer has shown that if the impeller has less diameter than the casing, the value, V, must be corrected to give the real tangential velocity of the water. To avoid any confusion regarding conditions after the particles are thrown off by the prime mover, the writer proposes to build a casing around the impeller, so as to know definitely what water is and is not rotating. To be more specific, the energy given to the water by the prime mover is the important factor to be considered. What is done with the energy after it is developed is of no interest at present. The writer is only trying to prove that the most the prime mover can possibly give the water is $\frac{V^2}{2g}$ and not $\frac{V^2}{g}$.

In building the centrifugal pump, the purpose is to construct an apparatus with which to whirl water as efficiently as possible. Considering, first, the shut-off conditions when no water is pumped, suppose the water whirled is contained in a cylindrical box of a radius, r. Suppose that in this box there are radial vanes, quite close together. For convenience, put the box on a vertical axis as shown in Fig. 32(a).

Fill the box at atmospheric pressure and then seal it completely. Now the prime mover brings the water to a stated speed and then keeps it moving uniformly. The pressure developed according to well-known laws will be atmospheric at the center and will attain a maximum at the outside. Taking the center as an origin and plotting pressure against radial distance, a parabolic curve is obtained as shown in Fig. 32(b).

A careful study of conditions in the closed box will throw considerable light on the centrifugal pump. Consider the particle, w, of Fig. 32(a) after the cylinder has attained its constant speed. In Fig. 32(c), the particle, w, is shown as a free body with the radial forces acting on it.

The radial forces acting will be: P, caused by the water pressure back of the particle, w; C, caused by inertia due to centrifugal force, of the col-

umn of water between w and the axis of rotation; and dC, caused by the inertia due to the centripetal acceleration of the particle, w.

P will always be greater than C by an amount just sufficient to keep the particle, w, traveling in a circular path. This is the important factor in the writer's argument. Particle w must be forced to travel in a circle; it will not do so otherwise. However, it travels in a circle only as long as the difference between P and C is dC (Fig. 32(c)). This will hold absolutely true as long as P and C follow the parabolic pressure distribution shown in Fig. 32(b).

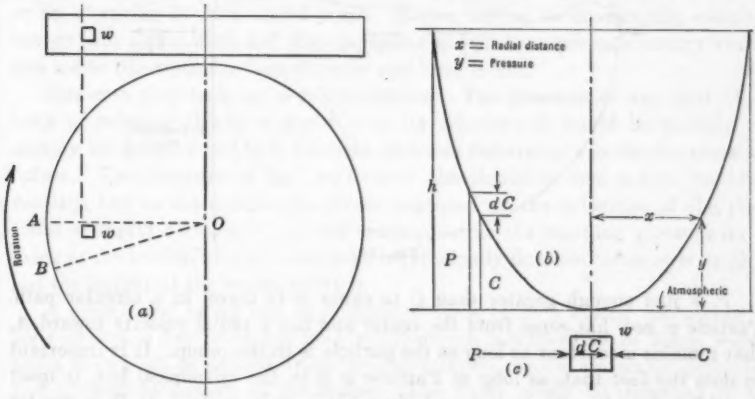

FIG. 32.

Now, suppose that, by some means, it were possible to give w a slight radial velocity, V_r, toward the center of the cylindrical tank. Particle w, now having all radial forces balanced, then moves toward the center of rotation, O. Having more tangential velocity than it needs for the new position, it is slowed up tangentially by pressing on Vane AO (Fig. 32(a)). At the same time the particle taking the position formerly occupied by w must be speeded up tangentially. Therefore, it presses against Vane BO so that the prime mover is not affected. The radial forces are still perfectly balanced on the particle, w, although P, C, and dC have all changed in value; w continues on its way to the center at this constant velocity. To get a clear idea of this behavior, consider the actual water surface to take the form of the paraboloid of revolution (Fig. 32(b)) and then, putting in the guide-vanes, consider the motion of the particle, w, in the form of a wood block having the density of water and acting under similar initial radial velocity conditions at the water surface.

Conclusions to Be Drawn from Shut-Off Condition.—Neglecting losses and assuming perfect guide-vanes, a particle of water is free to move radially. Given a radial velocity, V_r, that velocity will be maintained as long as the parabolic pressure distribution, originally assumed, exists.

Now, suppose water is admitted at the center of the pump with whatever radial velocity, V_r, is needed to supply the demand. Then a series of holes is opened all around the periphery of the cylindrical tank. The prime mover

maintains the constant speed necessary to make input equal to output. As long as the pump remains full and slip is neglected (which will be the best condition attainable), the pressure distribution within the cylindrical box will have the parabolic distribution. In any position, the forces acting upon the particle, w (P, C, and dC), are balanced just as they were in the box at shut-off.

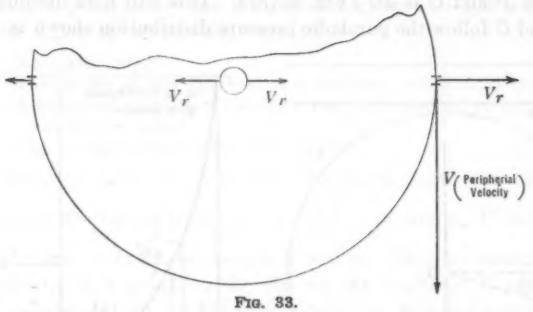

FIG. 33.

P is just enough greater than C to cause w to travel in a circular path. Particle w now has come from the center and has a radial velocity toward A, that remains a constant as long as the particle is in the pump. It is important to note the fact that, as long as Particle w is in the cylindrical box, it must travel in a circle. Since it travels in a circle only as long as P is greater than C, it follows that P is necessary to keep the particle in the box and after it leaves, of course, P, C, and dC all vanish together. Therefore, the writer believes that, after a careful unbiased study, it will be agreed the particle, w, entered with a radial velocity, V_r, and left with identically the same radial velocity.

Based on this discussion, the energy considerations are quite simple. The velocity of the jet issuing from the cylindrical box (Fig. 32(a)) will be as shown in Fig. 33.

The total energy contained in the water is obviously $\frac{V_r^2}{2g} + \frac{V^2}{2g}$, but $\frac{V_r^2}{2g}$ was given to the water before it entered and must be deducted from the output to give that furnished by the prime mover.

Some will say that the pressure head at the point of discharge has been neglected. This pressure was needed to keep the particles in the box in their circular paths.

Another point of view on energy is that particles entering with radial velocity, V_r, and no tangential velocity, are given a tangential velocity, V, by the prime mover, or the input is $\frac{V^2}{2g}$.

The purpose of allowing the casing to rotate is simply to place definite limits on the subject-matter. The energy given to the water by the prime mover is the important factor.

LLOYD H. DONNELL,* ESQ. (by letter).—The author brings up the interesting question as to whether, in a rotating body of liquid, there is both kinetic energy due to its motion and potential energy due to its pressure, or whether these are merely two successive steps in the transformation of the same energy from one form to another.

At first, the latter view does not seem unreasonable. It is natural to feel that the pressure is due solely to the motion and that this pressure would instantly disappear if the velocity were destroyed by any means, by friction or by changing it into useful work. Hence, trying to change the velocity energy into useful work and then expecting to still have pressure energy available seems like trying to "eat the cake and have it too."

However, this rests on a misconception. The pressure of any part of a body of rotating liquid is not due to its velocity; it would be possible to destroy its velocity and still have the pressure remaining exactly the same as before. The pressure of any particle of the liquid is due, not to its own velocity, but to the centrifugal forces produced by the velocities of the particles of liquid between it and the center, just as the crushing pressure on a brick at the bottom of a pile of bricks is principally due, not to its own weight, but the weight of the bricks above it.

A centrifugal pump can be considered roughly as a casing containing a body of liquid kept rotating by a motor-driven paddle-wheel or rotor, water being drawn off as desired from the outer layers, which are under pressure, and allowed to flow in at the center. The water might be imagined to be divided along the radius into thin layers. The outermost layer flows out through the discharge pipe and the next layer moves up to take its place, etc.

The point to be noted is that the tangential velocity of each successive outermost layer could be completely destroyed just before it was discharged, without affecting its pressure (as this pressure is due to the centrifugal force on the inner layers and only negligibly to the centrifugal force on the outermost layer itself, if the layers are assumed to be thin enough). The velocity could be destroyed by friction with the casing wall (as must actually be the case, to a large extent, at very low discharge rates), converting its energy into heat; or, theoretically, it could be changed into useful work with a set of miniature water-wheels. Thus, water would be discharging under the full pressure head and at the same time the kinetic energy of all the water discharged (no matter how thin each successive layer was assumed) would be converted into heat or some other separate form of energy.

Hence, there is, in the casing, both pressure energy (of which full use is made) and kinetic energy, which must have been paid for at the drive shaft, but which there is evidently some difficulty in utilizing. Some of it can be used as velocity in the discharge pipe, especially at high discharge rates when this velocity is comparable to the tangential velocity in the casing (hence it is possible to get high energy efficiencies at full discharge). However, especially at low discharge, much of the velocity energy must be changed into pressure energy, or else it will probably be wasted in eddy currents.

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It is possible to change velocity energy into pressure energy efficiently, by providing an expanding closed path in which the velocity gradually decreases as the pressure increases, as in a Venturi meter. It is difficult to understand how a spiral casing provides any such thing, because the expanding passage formed is entirely open all along one side and the pressure kept more or less constant all along this side. However, a closed expanding passage can be provided, either outside the casing with the small end forming the discharge opening of the casing, or in the form of a number of small passages arranged around the rotor inside the casing. To be effective, however, the velocity inside such a passage at the small end (which depends on its area and the discharge rate) must be the same as that of the water just outside the passage (which depends largely on the speed of rotation of the pump). With a fixed pump speed this can be the case only at one particular rate of discharge. Such a passage will be ineffective or even harmful at other rates of discharge—unless its size can be varied by some mechanical means.

S. TIMOSHENKO,* ESQ. (by letter).—This paper contains in several places a severe criticism of the usual theory for centrifugal pumps. The accepted theory is based on the assumption of an "ideal fluid" and on the neglect of various kinds of losses of energy; hence it is quite natural to expect considerable discrepancy, between the theoretical and experimental results. Although this discrepancy is very large the writer does not agree with Professor Sherzer that the causes are fundamental errors in the present theory, such as an erroneous addition of the "pressure head" to the "velocity head" or a neglect of the pressure in the casing which prevents the free radial acceleration.

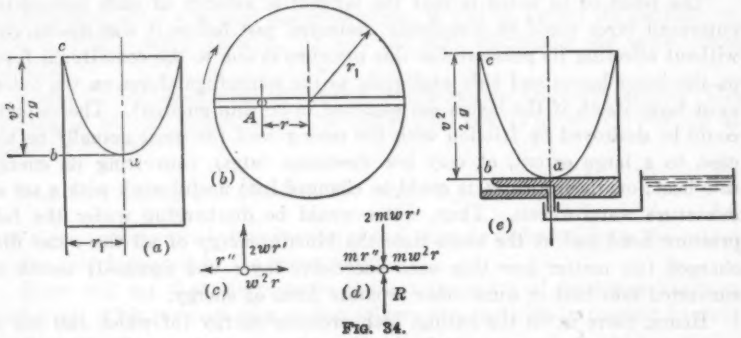

FIG. 34.

The technical part of the paper contains very interesting experimental results and applications of these results in design. However, the writer wishes to discuss only that part in which the usual theory is criticized. By use of two simple examples, it may be shown that the "velocity head" and the "pressure head" are not "part and parcel of each other," but are separate and distinct quantities, both of which can be utilized under certain conditions. In

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the usual theory, the pressure in the casing does not affect the amount of work done on the water in the radial grooves of the runner. Consider, first, a fluid rotating in a circular cylindrical container with a constant angular velocity, w (Fig. 34 (a)). It is well known that the surface of the rotating fluid will be a paraboloid of revolution. At the center, a, the velocity of the fluid and the pressure are both zero. At the point, c, the pressure is also zero, but the velocity is $v = wz_1$ and kinetic energy of 1 lb. of the fluid at this point will be $\frac{v^2}{2g}$. At the point, b, the kinetic energy and the so-called "pressure energy" are both equal to $\frac{v^2}{2g}$.

Consider, as a second elementary problem, the motion of a ball in a smooth groove of a rotating disk. For simplicity, assume that the groove is radial (Fig. 34(b)), that there is no friction, and that the ball can move freely along the groove. In order to investigate the motion of the ball, apply the principle of d'Alembert. On this basis equations for bodies in motion can be obtained in the same manner as equations for bodies in equilibrium, it being necessary only to add the inertia forces to the external forces. The inertia forces will be obtained by multiplying the components of acceleration with -1 and with the mass of the moving ball. In the present case, the complete acceleration of the ball will consist of three parts: (a) The acceleration of the ball in its relative motion along the radial groove, which will be represented by the second derivative, r'', of the radial distance, r, of the ball with respect to time; (b) the acceleration of A the instantaneous point of contact between the ball and the disk; in the case of uniform rotation, this is given by the magnitude of the centrifugal acceleration, w^2r; and (c) Corioli's acceleration, $2wr'$, which is perpendicular to the relative velocity, r' in the direction of motion. These three components of acceleration and the corresponding inertia forces are shown in Fig. 34(c) and (d). In addition to inertia forces, the external forces acting on the ball should be considered. Neglecting the weight of the ball, only one force remains—the reaction, R, of the groove, which produces the work on the ball. The differential equations of motion, (Fig. 34(d)), will be:

$$mr'' - mw^2r = 0 \quad \ldots\ldots(52)$$

$$R = 2mwr' \quad \ldots\ldots(53)$$

Equation (52) determines the motion of the ball along the groove. It may be written in the form:

$$r = r_0 \cosh w\,t + \frac{r_0'}{w} \sin w\,t \quad \ldots\ldots(54)$$

in which, r_0 denotes the initial displacement, and r_0' the initial relative velocity of the ball. Consider only the particular case, $r_0' = r_0 w$; that is, the ball at the initial instant has its radial and circumferential velocities equal. Under such an assumption:

$$r = r_0 \,(\cosh wt + \sinh wt) \quad \ldots\ldots(55)$$

$$r' = r_0 w \,(\cosh wt + \sinh wt) = rw \quad \ldots\ldots(56)$$

The radial component of the velocity remains equal to the tangential component during the motion of the ball in the groove, and the ball will leave the disk with the total velocity equal to,

$$\left(\sqrt{(r')^2 + w^2 r^2}\right)_{r=r_1} = \sqrt{2}\, w r_1 = \sqrt{2}\, v$$

in which, v denotes the circumferential velocity of the disk. The kinetic energy of the ball is equal to,

$$E = m v^2 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (57)$$

Neglecting the initial value of this energy, and assuming that the ball has been introduced at the center of the disk, the kinetic energy, Equation (57), must be equal to the work done by the reaction, R, which is the only force acting on the ball. In calculating this work, Equation (53) will be used. The reaction, R, is always perpendicular to the radius, and its moment with respect to the center of the disk will be $2\, m\, w\, r'\, r$. The work done by this moment per element of time, $d\, t$, will be $2\, m\, w^2\, r\, d\, r$, and the complete work produced on the ball during its motion from $r = 0$, to $r = r_1$, will be:

$$W = \int_0^{r_1} 2\, m\, w^2\, r\, d\, r = m\, v^2 \ldots\ldots\ldots\ldots\ldots\ldots (58)$$

which agrees with Equation (57). Imagine now that the ball is no longer free to move along the groove, and that a radial force, F, directed always toward the center of the disk, opposes the free motion of the ball in the groove. It is easy to see that in such a case Equation (54) remains unchanged and Equation (52) must be replaced by:

$$m\, r'' - m\, w^2\, r + F = 0 \ldots\ldots\ldots\ldots\ldots\ldots\ldots (59)$$

The relative velocity of the ball will depend now on the magnitude of the force, F. When this force is very small and can be neglected, Equation (52) is sufficient. When the value of F approaches $m\, w^2\, r$ (the centrifugal force), the relative velocity, r', approaches zero, the total velocity of the ball leaving the groove approaches v, and the kinetic energy of the ball will be $\frac{m\, v^2}{2}$; that is, only one-half the kinetic energy obtained for the case of the free relative motion of the ball in Equation (57). Then, considering the work done on the ball during this very slow motion along the groove, it is easy to see, from Equation (53), that the reaction, R, of the groove diminishes in the same proportion as the relative velocity, r', but the time of action of this force increases in the same proportion and, finally (Equation (58)), the work done on the ball does not depend on the relative velocity, r', and is always equal to $m\, v^2$. In this case, however, only one-half this work is transformed into the kinetic energy of the ball and the other half is spent on work against the pressure, F.

In order to apply this discussion to the study of pumps, unite the two problems represented in Fig. 34(a) and (b), into that shown in Fig. 34(c). The circular cylinder rotates about its vertical axis with a constant angular velocity, w, such that the parabolic surface of the liquid is tangent to the bottom of the container at the center, a. The bottom of the container consists of a circular disk with a smooth radial channel, $a\, b$, and provision is made for bringing the liquid to the center, a, of the disk with zero pressure and

zero velocity. When the opening, b, is closed the same conditions occur as in the case shown in Fig. 34(a), and if friction is neglected, rotation with constant angular velocity will proceed without the introduction of any work from the outside. Imagine, now, that there is an opening at b such that the liquid is moving slowly along the channel, $a\,b$, and that the depth of the container is a little less than $\frac{v^2}{2g}$. Then, for each pound of fluid brought (at zero pressure and zero velocity) to the point, a, 1 lb. of liquid will be delivered at the point, c, where the velocity is equal to v and the pressure is equal to zero. From the discussion of Fig. 34(b), it is clear that the work done on each pound of liquid during its motion along the channel, $a\,b$, is equal to $\frac{1}{g}v^2$. This work will be spent partly in increasing the kinetic energy of the fluid and partly in overcoming the pressure.

Considering now the extreme case of a very slow motion in the channel it is found, as before, that one-half the work will be transformed into kinetic energy, $\frac{v^2}{2g}$, and the other half into the so-called "pressure energy." In this manner there occurs at the point, b, a "velocity head" equal to $\frac{v^2}{2g}$ and a "pressure head" equal to $\frac{v^2}{2g}$. Both these parts have a definite physical meaning, namely, for each pound of liquid introduced at the point, a, there will be obtained a pound of liquid at c, at which point it has the kinetic energy, $\frac{v^2}{2g}$ and the potential energy, $\frac{v^2}{2g}$. The sum of these two forms of energy is equal to the work, $\frac{v^2}{g}$, done during motion of the liquid along the channel, $a\,b$, which work is exactly the same as in the case of the free radial acceleration, which occurs in pumps without casings and discharging in the air.

By a special arrangement, the kinetic energy of liquid at the point, c (Fig. 34(c)), can be utilized and if there are no friction losses or other sources of dissipation of energy, the liquid will be raised to the level of $\frac{v^2}{g}$ given by the usual theory.

Thus, it is evident that there are no fundamental errors in the usual theory. It will give correct results provided the assumptions on which it is derived are fulfilled. In actual pumps, if such a level as $\frac{v^2}{g}$ is not attained, it must be attributed to various kinds of losses in energy which are not taken into consideration by this theory, and to other deviations from the assumptions made. It is assumed, for instance, that in its relative motion the fluid has a velocity uniformly distributed over the cross-section of the groove and that

this velocity has a radial direction. More detailed investigations* show that the distribution of the velocity over the cross-section is not uniform. Velocity is smaller at the side of the groove at which pressure on the liquid is produced and larger on the opposite side. The same investigations show also that in a radial groove the relative velocity of the fluid near the periphery is no longer in a radial direction, and that it has a component in a circumferential direction. These deviations from the assumptions made in the usual theory must affect the final result, and may yield some explanation of discrepancies between theoretical and experimental values.

A. B. Cox,† Esq. (by letter).—Any one who has given any study to the history of science and engineering cannot but be impressed with the way in which new theories and practices have been—and are—constantly overthrowing old theories and practices, only to be themselves overthrown by still other theories and practices later. A thousand years or so from now, practically all the present theories and practices will certainly have been superseded or radically changed; but why wait a thousand years?. Why not encourage—rather than discourage—such men as Professor Sherzer, who are trying, now, to discover the errors in old theories and to devise others, better and more workable?

In Fig. 35 is shown a series of head-capacity curves of a steam-condenser circulating water pump with low head and high capacity, and in Fig. 36 a series for an oil-cooler circulating water pump with high head and low capacity. A notable feature of the curves of Fig. 35 is the V, or notch, near the middle of each curve. This droop in the curve was not noticed during the test until the last curve was being taken—at 720 rev. per min.—whereupon the points at this speed were carefully checked and found to be correct. It was then discovered that similar notched curves could be drawn through the points of all the other curve points taken at the lower speeds. The droop in the head-capacity curve appears to be characteristic of centrifugal pumps and fans.‡ Probably the chief reason that this peculiarity has not been noticed before is that most pumps and fans have only a slight droop in the head-capacity curve which is scarcely noticeable, and the points of the curve are usually taken on test only at the shut-off point and near the rating point, the section in which the V is most likely to occur nearly always being omitted.

The diameters of the pump runner of Fig. 35 were 19 in. outside and 14⅞ in. inside, and that of Fig. 36, 11⅝ in. and 5½ in., respectively. The values of the head at shut-off, as calculated by the formula, $\frac{V^2}{g}$, for the two pumps, are 110.6 ft. and 245 ft. at speeds of 720 and 1 750 rev. per min., respectively, instead of 33.5 ft. and 117.5 ft. as actually obtained on test. The values calculated by the formula, $\frac{V^2}{2g}$, are 55.3 ft. and 122.5 ft., respectively.

* See, for instance, W. Kacharski, "Stroemungen einer reibungs freien Fluessigkeit bei Rotation fester Körper," 1918, and W. Müller, "Allgemeine Sätze über die Strömung in Kreiselzädern und Turbinen," *Zeitschrift für angewandete Mathematik und Mechanik*, Bd. 7, 1927, s. 347.

† Designer, R. D. Nuttall Co., Pittsburgh, Pa.

‡ "Performance of Centrifugal Fans for Electrical Machinery," by C. J. Fechheimer, *Transactions*, Am. Soc. Mech. Engrs., Vol. 46 (1924), p. 287.

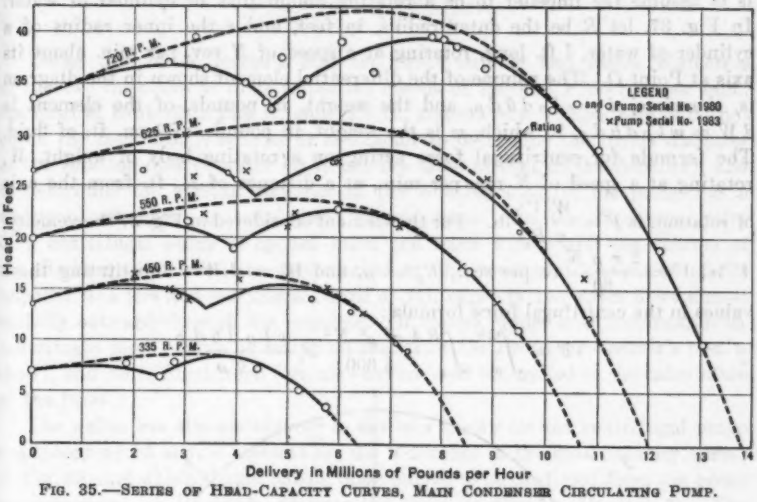

FIG. 35.—SERIES OF HEAD-CAPACITY CURVES, MAIN CONDENSER CIRCULATING PUMP.

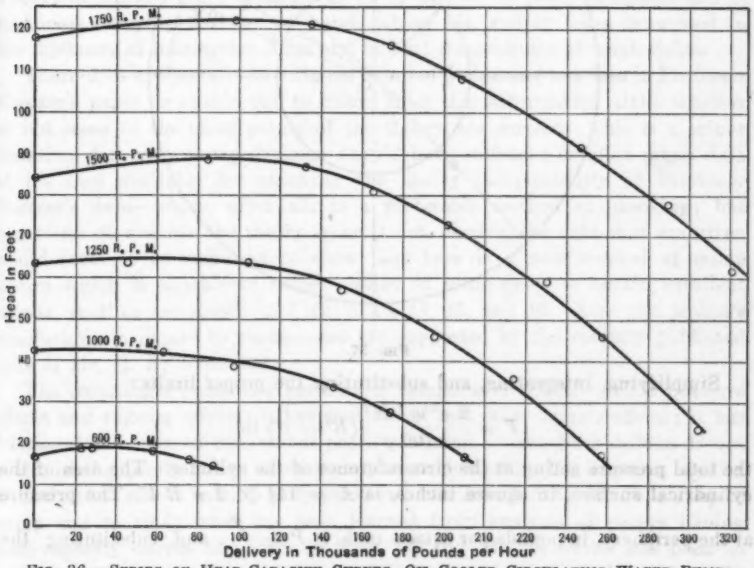

FIG. 36.—SERIES OF HEAD-CAPACITY CURVES, OIL-COOLER CIRCULATING WATER PUMP SERIAL NO. 1984.

Another way of considering the action of a centrifugal pump at shut-off is to assume the impeller to be a rotating hollow disk or cylinder of water. In Fig. 37, let R be the outer radius, in feet, and r the inner radius of a cylinder of water, l ft. long, rotating at a speed of N rev. per min. about its axis at Point O. The volume of the differential element shown in the diagram is, therefore, $dv = l\,\rho\,d\theta\,d\rho$, and the weight, in pounds, of the element is $dW = w\,l\,\rho\,d\theta\,d\rho$, in which, w is the weight, in pounds, of 1 cu. ft. of fluid. The formula for centrifugal force acting on a rotating body of weight, W, rotating at a speed of N rev. per min., at a distance of R_c ft. from the axis of rotation, is $F = \dfrac{W\,V^2}{g\,R_c}$ lb. For the element considered in Fig. 37 the velocity, V, is, $V = \dfrac{2\,\pi\,\rho\,N}{60}$ ft. per sec., $R_c = \rho$, and $W = d\,W$. Substituting these values in the centrifugal force formula:

$$F = \int_0^{2\pi} \int_r^R \frac{4\,\pi^2\,\rho^2\,N^2}{3\,600} \times \frac{w\,l\,\rho\,d\,\theta\,d\,\rho}{g\,\rho}$$

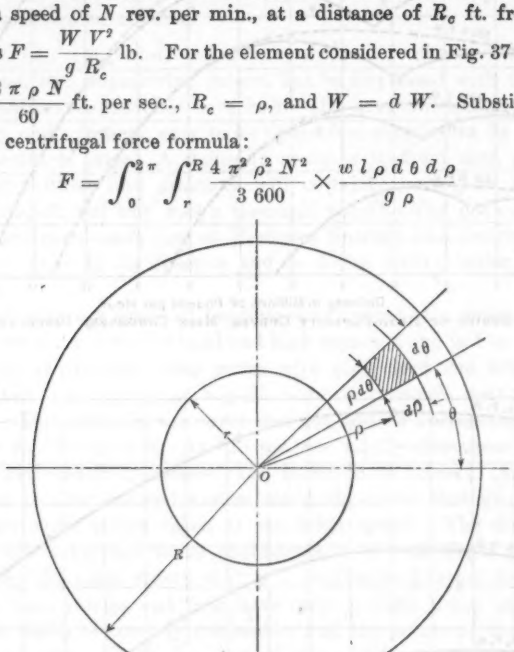

FIG. 37.

Simplifying, integrating, and substituting the proper limits:

$$F = \frac{2\,\pi^3\,w\,l\,N^2}{2\,700\,g}\,(R^3 - r^3)\text{ lb.}$$

the total pressure acting at the circumference of the cylinder. The area of the cylindrical surface, in square inches, is $A = 144 \times 2\,\pi\,R\,l$. The pressure at the periphery, in pounds per square inch, is $P = \dfrac{F}{A}$, and substituting the values of F and A,

$$P = \frac{\pi^2\,w\,N^2}{144 \times 2\,700\,g} \times \left[\frac{R^3 - r^3}{R}\right]$$

and the head, in feet, is,

$$H = \frac{\pi^2 N^2}{2\,700\,g} \times \left[\frac{R^3 - r^3}{R}\right]$$

Using this latter formula and the exact outside and inside diameters of the two impellers at their respective speeds of 720 and 1 750 rev. per min., the head at shut-off is 29.4 ft. and 72.8 ft., respectively. These results do not check the tests either, because it is difficult to determine the inside diameter of the rotating disk of water (it is not the same as that of the impeller), and to determine the outside diameter of the rotating disk, if the pump has a whirlpool chamber—as both these pumps have. As the discharge valve of a centrifugal pump is opened more and more widely and the volume of the discharge increases, the water takes up the rotational movement of the impeller to a less and less degree, until at full capacity the water flows almost radially outward through the impeller. Of course, under such circumstances, centrifugal force cannot be acting on the water (as Professor Sherzer's results show), and centrifugal force formulas cannot well be applied to the calculation of the head.

The writer has also endeavored to devise a theory for the centrifugal pump —a theory which should account for the V-notches in the head-capacity curves of Fig. 35, and which should enable these curves to be predicted from the pump measurements, as Professor Sherzer has done in Figs. 9, 10, and 11. The writer's endeavor has not yet been successful, but it has resulted in "A New Theory of Fluid Flow",* which may be of interest to pump designers and to engineers studying the flow of water, oil, or air, and to those interested in the thickness of lubricating films and in heat transmission through fluids.

There does not seem to be sufficient detail of design and test data in Professor Sherzer's paper to enable one to check from this information alone whether or not some of the chief points of the theory are correct. This is a minor omission, for every pump designer should have sufficient detailed pump data of his own available for checking the theory independently of Professor Sherzer's data—which, after all, is a preferable method of checking; but regardless of whether the theory is, or is not, correct, the data that are given would seem to be sufficient to show that here is a new method of pump design which is capable of being applied in some cases to obtain excellent results, such as are shown in Figs. 9, 10, 11, 15, and 16. Also, the author's conclusions in regard to guide-vanes are supported by the recently published tests of Mr. H. F. Schmidt.†

The centrifugal pump is a machine of complicated action. It has pressure effects and velocity effects; it has centrifugal and other inertia effects; it has frictional and eddy effects; and if poorly designed, it may have sudden droops in the head-capacity curve, or vacuum pockets formed in the impeller, similar to cavitation effects in ship propellers, etc. In an effort to design a simpler pump and to apply what has been learned from analysis of pumps having head-capacity curves as shown in Figs. 35 and 36, the writer has been so

* *Journal*, Franklin Inst., December, 1924, and December, 1926.

† "Some Screw Propeller Experiments with Particular Reference to Pumps and Blowers," *Journal*, Am. Soc. of Naval Engrs., February, 1928.

fortunate as to have developed a radically new but simple type of axial flow pump. The first tests indicate that the weaknesses typical of the usual design of this type of pump have been avoided and the good points of the centrifugal pump—particularly for high-head work—have been retained and improved upon.

ALLEN F. SHERZER,* ASSOC. M. AM. SOC. C. E. (by letter).—The writer wishes to record his appreciation of the very intelligent discussion accorded his paper. An analysis of the response reveals two interesting facts. First, as might be expected, any attempt to question the logic and accuracy of the present pump theory was keenly resented by many readers and was often regarded as the height of presumption. On the other hand it was gratifying to note that many readers, while skeptical at first, began to apply the new theory to pumps of their own design with quite satisfactory results. It is believed that as time goes on others may feel inclined to adopt this method of analysis in principle at least. It should be regarded in the same light as other hydraulic investigations in that factors and coefficients from practice are essential, and these are modified by proportions and other features of the design. Knowledge of the flow over weirs, through orifices, and in channels rests on the same sort of foundation. There are many variables and they are difficult to separate.

The second feature of interest brought out by the discussion is that those who contributed to it are often quite unable to agree among themselves as to what is the correct theory for the centrifugal pump. Some maintain what they term "the old established theory" to be correct, others say its errors have been recognized long ago, and that it has been replaced by new theories. Some say the head at shut-off is $\frac{V_2^2}{g}$, others that it is equal to, or less than, $\frac{V_2^2}{2g}$. In general, it may be said that most of the facts claimed as correct by one contributor are denied by another, and the chief point in common is their disagreement with the writer. It is gratifying that nothing more serious than this was raised against the new theory.

In order to avoid useless repetition several of the points to which most frequent objection was taken will be singled out for special attention and discussed first. Following this, an attempt will be made to reply adequately to each discussion contributed.

The four points subject to most general criticism were: First, the question of adding the pressure and velocity heads in order to obtain the total head in the theory of the centrifugal pump; second, the question as to the correctness of Equations (*B*) and (*C*) as applied to the centrifugal pump; third, the logic of the three-orifice theory; and fourth, the application of the three-orifice theory to pumps having rising characteristics. These will be discussed in the order given.

The question of adding the pressure and velocity heads to obtain the total head supplied to the water in a centrifugal pump seems to have been the cause

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of most of the objection to the new theory. The writer stated emphatically that it is incorrect to regard these two as separate and distinct amounts of energy in studying the action of a centrifugal pump. The pressure developed by such a pump is due, as the name implies, to centrifugal force; and that force is not, and never was, energy. It seems incredible that so many are apparently unable to distinguish correctly between force and work.

Another fact that seems to have escaped general attention is that the existence of pressure in an incompressible fluid like water does not at all imply the presence of energy. Consider, for example, a strong steel box entirely filled with water under a pressure of 100 lb. per sq. in. Neglecting the elasticity of the box and the compressibility of the water, this pressure, which has a definite existence, and can easily be measured, has no significance whatever as energy. Nothing useful can be done with it. If a small valve in the box is opened no water would flow, and the pressure would have disappeared.

Under some conditions pressure may be regarded as energy, but not always. Messrs. Blaisdell and Timoshenko both devote some space to this question, but conclude that in the pump theory it is correct to add the velocity head to the pressure head produced by it to get the total head given to the water. They submit diagrams purporting to show how this may be done. The main difficulty with their arguments is that the machines they describe are not centrifugal pumps as generally known. Their conclusions are perfectly correct for the machine they are considering, but not for the modern centrifugal pump.

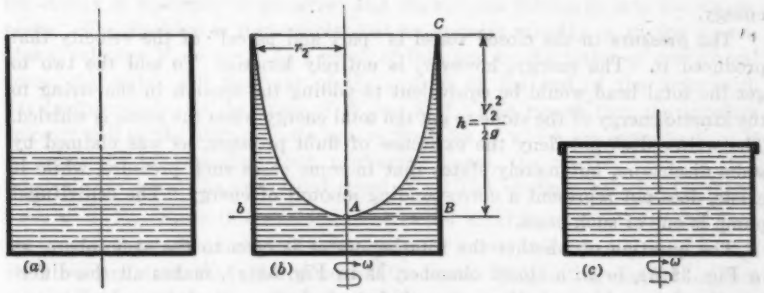

FIG. 38.

An illustration will make this clear. Assume a cylindrical tank free to rotate about its central axis and partly filled with water, as in Fig. 38 (*a*). If this vessel is set in rotation with an angular velocity, ω, the surface of the water, as is well known, will take on the shape of a paraboloid of revolution (see Fig. 38 (*b*)). At the center, (*A*), both the velocity and pressure are zero. Considering a pound of water at Point *C*, and with reference to a horizontal datum plane through Point *A*, it is clear that since *C* is revolving with a peripheral velocity, $V = \omega R$, a pound of water there possesses a kinetic energy, $\frac{V^2}{2g}$. It is also evident that this same pound of water is located $h = \frac{V^2}{2g}$ ft. above

the datum plane through Point A and hence possesses $h = \frac{V_2^2}{2g}$ ft-lb. of potential energy. The total energy is, therefore, the sum of the two, or $\frac{V_2^2}{g}$. The writer is in entire agreement with this result, but would venture to point out that this is not the way centrifugal pumps are built.

In Fig. 38 (c) the cylindrical tank has been cut off at the water-line and a tight-fitting cover has been placed so that the tank is entirely filled with water. Now, if this closed tank is set in rotation with an angular velocity, ω, as in Fig. 38 (b), conditions are found to be entirely changed. In the case of the closed vessel less work is done on the water than when the surface is open. The reason is simple since in starting from rest, work is done in accelerating the water just as a fly-wheel is brought up to speed; and, in addition, the water (Fig. 38 (b)) has been given potential energy. In the closed vessel (Fig. 38 (c)) the only work done is in bringing the mass of water up to speed since no change of level takes place. The pressure at the circumference of the closed vessel is $\frac{V_2^2}{2g}$, or exactly the same as at Point B of the open vessel; but in the former case the pressure has no significance as energy and no work is done to maintain it. In the latter case the pressure is a measure of the potential energy, $h = \frac{V_2^2}{2g}$, and work was done to give the water this energy.

The pressure in the closed vessel is "part and parcel" of the velocity that produced it. The energy, however, is entirely kinetic. To add the two to get the total head would be equivalent to adding the tension in the string to the kinetic energy of the stone to get the total energy when the stone is whirled. The writer does not deny the existence of fluid pressure, as was claimed by some discussors, but merely states that in some cases such pressure, while it exists, does not represent a corresponding amount of energy. The centrifugal pump is one of such cases.

The question of whether the rotating water is open to the atmosphere, as in Fig. 38(b), or in a closed chamber, as in Fig. 38(c), makes all the difference in the world as to the amount of work done on each pound of water. It will be easily seen that conditions in the centrifugal pump correspond to Fig. 38(c) and not to Fig. 38(b). A particle of water at the outer circumference (Fig. 38(c)) has both pressure and velocity heads, but its energy is only kinetic. To state in the general pump theory that conditions are as illustrated in Fig. 38(b) would be about the same as saying that in considering a particle of water below the surface of a lake the pressure due to its depth is added to the depth which produced the pressure to get the total head.

The second point to which considerable objection was made was the writer's statement that his Equations (B) and (C) were not applicable to the centrifugal pump. Several discussors have insisted that Equation (B) was the only one that could satisfactorily explain the performance of a centrifugal pump; others preferred Equation (C), or some simple modification of it. Some

remarked that the equations must be used by one who understands what they mean—the idea being that every one was not so qualified. Judging solely from these statements, the writer doubts whether any of those discussors have a clear conception of the actual meaning and physical significance of Equations (*B*) and (*C*). Equation (*B*), while correct in principle, at least for a turbine, can be eliminated at once from consideration as applied to a centrifugal pump. The reason for this is given on page 6 of the paper and may be found elsewhere in texts and works of reference. Equation (*C*), therefore, remains to be discussed.

Referring to Fig. 1 it is seen that,

$$C_2 \cos \alpha_2 = V_2 - W_2 \cos (180° - \beta_2) = V_2 + W_2 \cos \beta_2$$

Substituting this value of $C_2 \cos \alpha_2$ in Equation (*C*):

$$H = \frac{V_2^2}{g} - \frac{V_2 W_2}{g} \cos (180° - \beta_2) = \frac{V_2^2}{g} + \frac{V_2 W_2}{g} \cos \beta_2 \ldots\ldots (60)$$

This will be recognized as Equation (37) so often referred to in the discussion by Professor Harris. The writer prefers to take an indirect method of showing the real physical meaning of Equations (*C*) and (60).

Consider a machine which might be termed a mechanically driven lawn sprinkler, as shown in Fig. 39. This consists of two radial arms attached to a central pipe by which they may be revolved about the center of the pipe as an axis. The ends of the arms are plugged and an orifice is made in each, as shown. The pipe and arms are filled with water and provision is made for keeping up the supply at atmospheric pressure; and the streams discharge into the atmosphere. The machine is then revolved at an angular velocity, ω, giving a peripheral velocity, $V_2 = \omega r_2$. As a result of this, and by the action of centrifugal force, the water will flow out of the orifices at the end of the arms with a velocity relative to the arms of $W_2 = V_2$.* This means that the absolute velocity will be the diagonal of a square the sides of which are equal to V_2. It is important to remember that, neglecting losses, the relative velocity, W_2, must always equal the peripheral speed, V_2. The absolute velocity is, therefore, $\sqrt{2}\ V_2$. Since this machine receives the water at zero velocity and zero pressure at the center and gives to it ultimately a final velocity, ($C_2 = \sqrt{2}\ V_2$), it is evident that the work done on the water (neglecting losses) is that required to produce the absolute velocity, C_2, and its energy is entirely kinetic and equal to $\frac{C_2^2}{2g}$, or $\frac{(\sqrt{2}\ V_2)^2}{2g} = \frac{V_2^2}{g}$ for each pound of water.

Now, assume a more general case in which the arms are not radial, but are curved forward as in Fig. 39(*b*). The only change in this case is that, while $W_2 = V_2$ as before, it now leaves at a different angle relative to V_2. The absolute velocity, C_2, is the vector sum of (V_2) and ($W_2 = V_2$). The work done is $\frac{C_2^2}{2g}$ as before, for each pound of water. By trigonometry,

$$C_2^2 = V_2^2 + W_2^2 - 2\ V_2\ W_2 \cos (180° - \beta_2)$$

or,

$$C_2^2 = V_2^2 + W_2^2 + 2\ V_2\ W_2 \cos \beta_2$$

* For proof, see p. 24.

The total head given to each pound of water is $\frac{C_2^2}{2g}$; hence,

$$H = \frac{C_2^2}{2g} = \frac{V_2^2 + W_2^2 + 2\,V_2\,W_2 \cos\beta_2}{2g}$$

By the conditions of the problem, W_2 must always be equal to V_2. Hence,

$$H = \frac{C_2^2}{2g} = \frac{2\,V_2^2 + 2\,V_2\,W_2\cos\beta_2}{2g} = \frac{V_2^2 + V_2\,W_2\cos\beta_2}{g}$$

This will be recognized as the transformed Equation (C), or Equation (37), as given by Professor Harris.

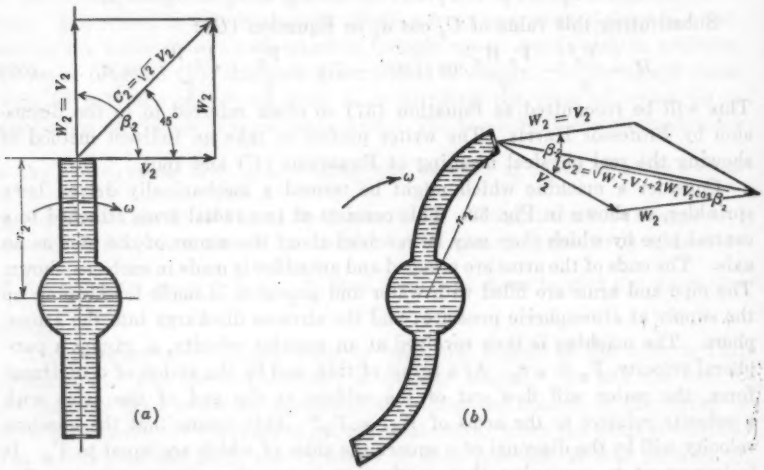

FIG. 39.

It should now be clear as to just what kind of machine these equations apply. It is certainly not a modern centrifugal pump; it resembles more a lawn sprinkler mechanically driven. Equations (C) and (60) apply only to such machines as have all their energy in the kinetic form at the point of discharge from the wheel. The theory of its operation is based on the assumption that the water is discharged against only atmospheric pressure. As neither of these conditions are even remotely realized in the centrifugal pump, it is not unreasonable to suspect that this difference in the assumptions might be the cause of the generally observed discrepancy between the ordinary theory and the actual performance of a pump.

At this point another error has come into the generally accepted pump theory and equations. It is well known that the quantity is a function of the head for a centrifugal pump at constant speed. Most texts attempt to prove this by reference to Equation (60), as follows: By the law of continuity of flow, $W_2 = \frac{Q}{A_2}$, in which, A_2 is the total area of the impeller perpendicular

to W_2. By substituting this value in Equation (60) the result is:

$$H = \frac{V_2^2}{g} + \frac{V_2 \cos \beta_2 \, Q}{g \, A_2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (61)$$

This is the usual straight-line equation so often seen in texts and generally mentioned as the ideal quantity head curve. The error here lies in the fact that, while $W_2 = \frac{Q}{A_2}$, W_2 is fixed by the conditions of the problem as always equal to V_2, and since A_2 is fixed by the design of the pump, it cannot be a variable and the whole equation is meaningless. In fact, it can be easily seen that with such a machine running at constant speed the flow would be a definite and fixed amount, and nothing but a change of speed could alter it. It is now evident that all these familiar straight-line curves are based on the same fallacy. It is small wonder that no better agreement has been obtained. Equations (*C*) and (60) are merely different expressions for the kinetic energy of the water discharged from the arms of such a machine as is shown in Fig. 39(*a*) and (*b*), but this simple relationship does not appear to be generally known. The present pump theory seems to be almost perfectly applicable to what might be called a kinetic pump the object of which is to give only kinetic energy to water. To make this clear it will be noted from Equation (60) that if $\beta_2 = 90°$, $H = \frac{V_2^2}{g}$; if $\beta_2 = 0°$, $H = \frac{2\,V_2^2}{g}$; and if $\beta_2 = 180°$, $H = 0$. All these results have been recently verified by tests on such a kinetic pump which demonstrates beyond any doubt the real meaning of those equations.*

The three-orifice theory has been the subject of much of the adverse criticism of the paper, largely, the writer believes, due to misunderstanding. Every centrifugal pump designer appreciates very well that the greater part of his calculations center about these three points and that almost the entire performance of a pump is determined by its proportions at these places.

The usual theory as well as those modified ones suggested in the discussion are invariably weak in that no attempt has been made to take any account of the action of the case in determining the pump characteristic. Any designer knows that a change in what the writer terms Orifice No. 3 (Fig. 8), will completely alter the performance of an impeller. This fact has been given little if any attention in the various opinions presented in the discussion. The effect of a change in the area of Orifice No. 3 is shown graphically by a comparison between Curves *A* and *B* in Fig. 40. This was obtained from an actual test of an impeller in two casings in which the area of Orifice No. 3 was varied. The writer fails to see how Equations (*B*) or (*C*), or Professor Harris' Equation (37), would explain this fact. It is well known to most designers that a change in Orifice No. 3 has two distinct effects: First, a change in the shut-off head; and, second, a change in the slope of the pump characteristic. To study the effect of the area on the slope of the curve the two have been adjusted so as to have the same shut-off head as in Curves *A* and *C*. It is easily seen that the smaller orifice, No. 3, gives the steeper curve, which is just as might be expected and in perfect agreement with the writer's theory. In other words,

* For test results covering this point see p. 98 *et seq.*

if it is small, a greater fall in pressure is necessary to obtain a given quantity of water through Orifice No. 3 than if it is large. The other effect in determining the shut-off head must be gained largely from experience and, while it might be liable to cause trouble for a beginner, any experienced pump designer would have sufficient data at hand to estimate this quite accurately. Beyond this point the theory given in the paper will apply.

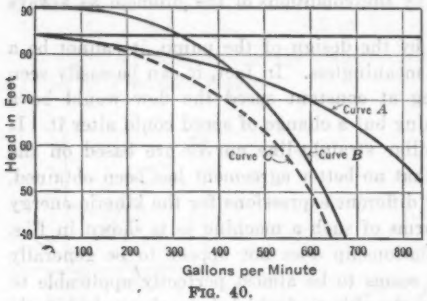

FIG. 40.

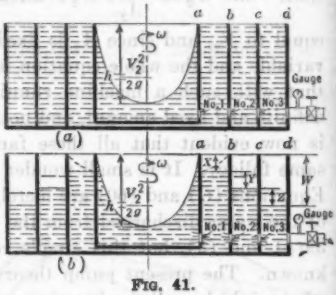

FIG. 41.

To make this clear, assume a machine, as shown in Fig. 41(*a*), which is the familiar rotating water paraboloid, Fig. 38(*b*), except that a hole has been made in the side (*a*) of the tank at the level of the vertex of the parabola (Orifice No. 1). Next, assume a stationary cylinder, (*b*), in Fig. 41(*a*), containing water, to be placed concentrically with the rotating cylinder, (*a*). If the water levels in Cylinders (*a*) and (*b*) are the same, no water will flow from Cylinder (*a*) to Cylinder (*b*) since the pressures on either side of Orifice No. 1 are equal and of an amount, $h = \frac{V_2^2}{2g}$. Next, assume a second stationary cylinder, (*c*), (Fig. 41(*a*)), to be placed as shown and a hole made in the side of the tank, (*b*), to form Orifice No. 2. If the water level in Cylinder (*c*) is that of Cylinders (*a*) and (*b*), no water will flow through Orifice No. 2. If a third stationary cylinder, (*d*), in Fig. 41(*a*), is provided and filled to the same level as Cylinders (*a*), (*b*), and (*c*), no water will flow from Orifice No. 3. Now, if a pipe with a valve is provided, as shown in Cylinder (*d*), and the pipe either leads to the atmosphere or some point where the pressure is less than $\frac{V_2^2}{2g}$, water will flow from Tank (*d*). This means that the level will fall in Tank (*d*) and water will flow in from Tank (*c*) through Orifice No. 3. This, in turn, will cause a drop in level in Tank (*c*) and water will flow through Orifice No. 2. Again, this will cause a drop in the level in Tank (*b*) and water will flow in from Tank (*a*) through Orifice No. 1. Of course, it is assumed that the water in Tanks (*b*), (*c*), and (*d*) is stationary and only Tank (*a*) revolves; also, that water is added to Tank (*a*) to make up for that discharged from Tank (*d*).

Fig. 41(*b*) shows the water levels in the tanks when flow is taking place. The drop to cause flow from Orifice No. 1 is shown at *X*, through Orifice No.

2 at Y and through Orifice No. 3 at Z. The total drop in pressure to cause flow through the pump is W, which is the sum of X, Y, and Z. As in the case of a centrifugal pump, the quantity of water allowed to flow is controlled by the discharge valve from Tank (d). When this valve is closed the tanks fill up to the level, $h = \frac{V_2^2}{2g}$, and this is the shut-off condition or point of no delivery. Then, if the discharge valve is opened a little some water will flow and the levels in Tanks (b), (c), and (d) will adjust themselves accordingly. The amount of the drop in level between the tanks is controlled largely by the area of the three orifices. The level in Tank (a) is, of course, always constant since it revolves at constant speed. If it were desired to express the relation between the flow of water through this pump and the reading of a pressure gauge located in the discharge line from Tank (d), that is, to draw the Q-H-curve of the pump—as it is usually called—the procedure would be as follows: When no water is discharged, the pressure gauge would read $h = \frac{V_2^2}{2g}$, and this may be represented by a horizontal line in Fig. 42. Next, draw the curves for the orifices as shown. Then add these together and obtain the combined curve. Next, subtract this curve graphically from the horizontal line, $H = \frac{V_2^2}{2g}$, and obtain the Q-H-curve for the pump. It should now be evident that this, in principle at least, is exactly what is taking place in a centrifugal pump although the details of design and construction may be different.

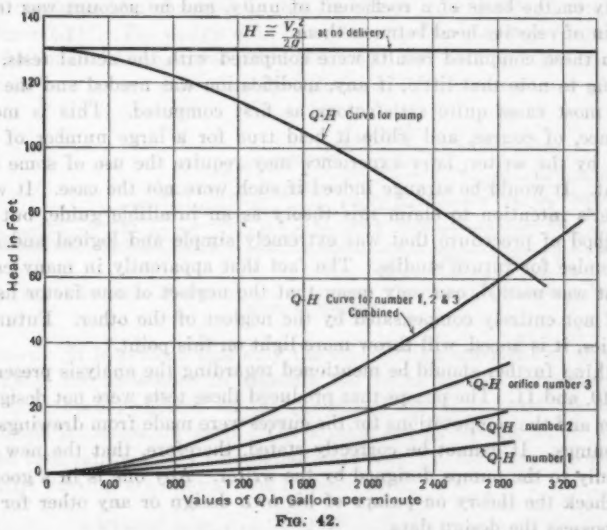

FIG. 42.

A closer approach to actual similarity would be to consider that the water in Tank (b) might be set in revolution by the dragging friction with Tank

(*a*), as well as by the jet-like action of the absolute velocity of the water flowing from Orifice No. 1. This would cause the surface in Tank (*b*) to be as shown in dotted lines in Fig. 41(*b*), and would have the effect of increasing the height of the shut-off head, thereby raising all points of the curve. This is often noticed in actual pumps. In fact, as several discussors pointed out, most of the better pumps exhibit this to a greater or less degree, and shut-off heads exceeding $\frac{V^2}{2g}$ are often obtained. This shows the elementary action of a centrifugal pump and that the shape of the final curve is determined to a large extent by the dimensions of the three orifices. No higher mathematics is needed and the result agrees well with practice, after a few simple coefficients are obtained.

One matter of criticism for which the writer must accept responsibility was his failure to explain fully the apparent neglect of velocity of approach in determining the drop through the three orifices. This was by no means ignored, but the full reasoning was not given. Having accepted the three-orifice theory as a matter of principle it was realized that the flow through any orifice is governed, among other things, by a coefficient of discharge. The difficulties in determining this coefficient for each of these orifices separately were obviously too great to hold out much hope for success. It was decided, therefore, to determine a blanket factor or coefficient by which to multiply the combined discharge of the three to bring the computed results into agreement with actual tests. Accordingly, the drop was computed for each orifice separately on the basis of a coefficient of unity, and no account was taken of the regain of velocity head between them.

When these computed results were compared with the actual tests, it was interesting to note that little, if any, modification was needed and the results were in most cases quite satisfactory as first computed. This is merely a coincidence, of course, and while it held true for a large number of pumps analyzed by the writer, later experience may require the use of some sort of coefficient. It would be strange indeed if such were not the case. It was not the writer's intention to claim this theory as an infallible guide, but rather as a method of procedure that was extremely simple and logical and showed great promise for future studies. The fact that apparently in many cases no coefficient was needed, can only mean that the neglect of one factor has been nearly if not entirely compensated by the neglect of the other. Future tests and studies, it is hoped, will throw more light on this point.

One thing further should be mentioned regarding the analysis presented in Figs. 9, 10, and 11. The pumps that produced these tests were not designed by the writer and the computations for the curves were made from drawings of the several pumps. It cannot be correctly stated, therefore, that the new theory applies only to the pumps designed by the writer. Any one is in a good position to check the theory on pumps of his own design or any other for which he may possess the design data.

It was claimed by several discussors that the new theory failed to account for pumps having a rising characteristic. This is not the case; but the claim

is possibly valid since such an analysis was not worked out in detail, being merely alluded to in the paper. Under the heading, "Effects of Casing", the writer mentions that the jet-like action of the water discharged from the impeller assists the general rotation of the water in the pump-case and thereby in some cases increases the head developed by the pump.

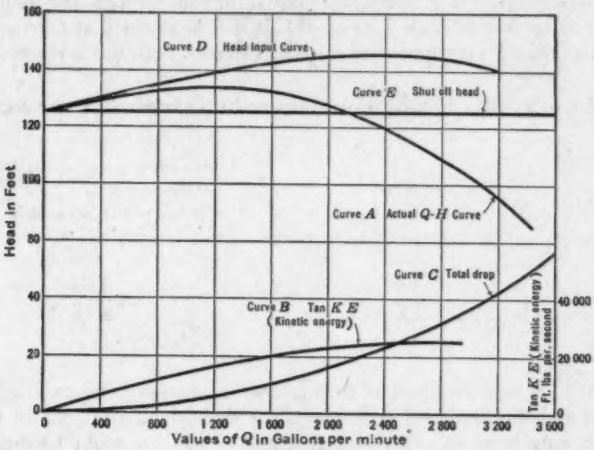

FIG. 43.

The characteristic obtained by test is shown in Fig. 43 marked "Actual Q-H-Curve (A)". The design data being known, the discharge diagrams were drawn for rates of discharge of 800, 1 600, 2 400, and 3 200 gal. per min. These are shown in Fig. 44. The absolute velocities, C_2, for these rates of flow are

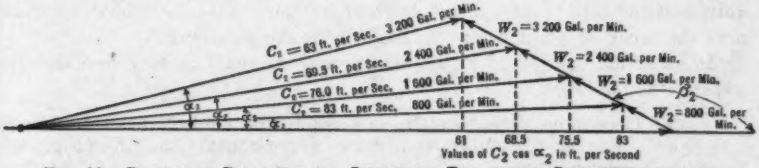

FIG. 44.—DISCHARGE DIAGRAMS FOR COMPUTING TANGENTIAL COMPONENT OF THE KINETIC ENERGY.

also shown. It is evident that only the tangential component of this velocity is available for assistance in rotating the water in the pump-case. These values have been projected on the peripheral speed, V_2, and the tangential components indicated for each. The kinetic energy of this water discharged into the case can be easily calculated since both the mass per second and the tangential velocity are known—the formula being,

$$\text{Kinetic energy} = \frac{1}{2} M V^2 = \frac{\gamma Q}{2g} (C_2 \cos \alpha_2)^2$$

The computations are arranged for clearness in Table 3.

Curve *B* (Fig. 43), represents the tangential kinetic energy as a function of the rate of flow through the pump. The total drop through the three orifices is shown combined in Curve *C*. If Curve *C* is added graphically to the Test Curve *A*, for purposes of analysis, Curve *D* is obtained. This represents the total head input to the water passing through the pump as a function of the rate of discharge. If a horizontal line is drawn through the shut-off head determined by test as Line *E* (Fig. 43), it will be noted that Curve *D* is in a relation to Line *E* very much the same as Curve *B* is to the horizontal axis.

TABLE 3.—KINETIC ENERGY COMPUTED FROM TANGENTIAL COMPONENT OF C_2.

Gallons per minute.	C_2, in feet per second.	Kinetic energy computed from tangential component of $C_2 = \frac{\gamma Q (C_2 \cos a)^2}{2g}$, in foot-pounds per second.
0	90.5	0
800	83.0	11 930
1 600	75.5	17 750
2 400	68.5	24 400
3 200	61.0	25 700

All that is now required is to establish a relation between the values of Curve *B* and the increased head above the shut-off point shown in Curve *D*. This can only be done experimentally, and it will no doubt be different for various designs and proportions. The writer has found from the analysis of a large number of such pumps that, for the ordinary pump and over most of the range of the characteristic, for every 1 000 ft-lb. per sec. of tangential kinetic energy, the head is increased about 0.8 ft. This varies somewhat and it is best that each designer determine such a value for himself, based on his usual designs and proportions. In the absence of such data 0.8 ft. will prove quite satisfactory. This will not hold, of course, for the section of the curve near the break-off point where the capacity is the maximum.

In an actual analysis of a pump the procedure would be very similar. The steps are as follows:

(1) Determine as accurately as possible the shut-off head (either by test or formula) and draw a horizontal line, *E* (Fig. 43), through it.
(2) Compute and plot Curve *B*.
(3) Apply the suggested coefficient of 0.80 per 1 000 ft-lb. per sec. to the values of Curve *B* and plot it above Line *E* as feet of head. This gives Curve *D*, or the total head curve.
(4) Calculate and plot Curve *C* showing the total drop through the three orifices given in the paper (page 18).
(5) Subtract Curve *C* from Curve *D* graphically and obtain Curve *A*, the approximate characteristic curve.

This is a very simple and direct method which the writer has used successfully for some time. Its accuracy rests entirely on the correct selection of the factor converting tangential kinetic energy into head. After a few such analyses are made, any one is in a position to determine that factor for himself.

It is not by any means constant over a single characteristic and, of course, varies in different designs. Pumps of unusual proportions will require special coefficients. One great advantage of this method is that it enables a designer to predict in advance whether a pump will have a rising, flat, or steep characteristic. It is easy to see that if the values of Curve C are greater than those of Curve D above Line E, then a falling characteristic will be obtained.

Referring to the point of no delivery, pumps which ordinarily have a flat or falling characteristic may appear to having a rising one on some tests because of the presence of air in the water or a slight leakage of air through the stuffing-box or suction line. This is due to the depression of the head at that point. Of course, the foregoing analysis will not fit such tests. Care should be taken to make sure that a reliable characteristic is obtained when accurate comparisons are desired. An error in the head of no delivery will be reflected in all the other values to the same degree. In many cases the tangential kinetic energy is so small that little if any effect is noticed, and it is sufficiently accurate to deduct Curve C directly from Line E. This was the case for the pumps analyzed in the paper and is perhaps the situation most frequently encountered.

Mr. MacMeeken attaches great importance to Equation (B) stating that it is the only one that can properly explain the action of a centrifugal pump. He further points out that no element must be omitted; but neglects to explain how the term, $C_1 V_1 \cos \alpha_1$, is obtained and this is the question that has given almost universal difficulty. Most other authorities disagree with him in preferring Equation (B) to Equation (C). Other discussors have denied the correctness of either. The writer has made inquiries regarding the special pumps mentioned which gave heads approximating $\frac{V_2^2}{g}$. From such information as was obtained it would appear that these were not centrifugal pumps at all, but were of the rotary type.

Mr. MacMeeken mentions the accepted and proved theory, which raises the point as to who accepts it and how it was proved. The statement that the writer's design will always require a larger impeller than a volute pump of good design is in direct contradiction to the data presented. Reference to Fig. 16 will clearly show the greater head developed by the circular case-pump compared with a good volute pump. Both impellers are of the same diameter and run at the same rotative speed. It is well known that volute pumps may develop greater heads than $\frac{V_2^2}{2\,g}$ and the same is true of circular case-pumps to an even greater degree.

Mr. MacMeeken's objection to Fig. 4(b) indicates a lack of understanding of the reasoning. The total power required to drive the pump may be divided into two parts, one to give energy to the water and the other to overcome the various losses in the pump (mechanical and otherwise). The total losses were determined by test as approximately 3.75 h.p., which left 1.25 h.p. as the energy given to the water. The sum of the two was 5.00 h.p., which was found by test to be required to drive the pump at a discharge of 50 gal. per min.

The 1.25 h.p. was found to be just what was required to lift 50 gal. per min. to the head of 100 ft. and this was checked by an actual test. This indicates a hydraulic efficiency of about 100% for small rates of flow. As a matter of fact it could not be otherwise. Assume, for example, that Mr. MacMeeken was correct and that the efficiency was less than 100% (say, 60 per cent.). Then, there would arise the interesting situation of putting into the water 0.60×1.25 h.p. $= 0.75$ h.p., and actually getting out 1.25 h.p. in useful work. This would be better than perpetual motion. When little or no water flows there are no great losses of energy and the hydraulic efficiency is nearly 100 per cent. Mr. MacMeeken states that, on the contrary, the losses will be very great. A few simple tests should show his error. The writer agrees with the transformation of Equation (B), but does not believe that,

$$\frac{C_2^2 - C_1^2}{2g} = \frac{W_2^2 - W_1^2}{2g}$$

Even if that relation is correct the situation is no better than before. The ratio of the dimension, A to C, as mentioned, affects the shut-off head which has been already considered.

Mr. MacMeeken claims that the so-called commercial efficiency of pumps mentioned in the paper is 77 per cent. The efficiency of 60% as given was determined by taking the average of ten quotations submitted by leading pump manufacturers, including the company with which Mr. MacMeeken was associated. This was the best information obtainable at the time the paper was written. However, no quotation as high as 77% was received.

It is true, that the circular case is not new as emphasized in the paper. Mr. MacMeeken's statement that the volute is superior to the circular case is contrary to easily proved facts. In the case of the multi-stage designs there were other reasons for the difference.

It is gratifying to learn that Mr. Kingsford applied the new theory to his own pumps with such success. It should be stated that the relative efficiencies of the two pumps compared in Fig. 16 were about the same. This is due to the fact that the head for the circular case-pump was more than 100 ft. greater than for the volute pump. When the circular case-pump impeller was reduced so as to give the same head as the volute pump, there were about five points of efficiency in favor of the circular-case design.

Mr. Moss raises the question as to who is qualified to pass on the meaning of the fundamental pump equations. From the discussion it is evident that he is one of the few who still believe in them. Most of the others believe that to be of any use at all they require considerable modification. Mr. Moss feels that not enough detailed equations are given in the paper. The fact is that none is needed except those that were given. The writer considers this simplicity rather an advantage than otherwise. Whatever else may be said about the theory it is certainly new and not at all similar to any found in current texts. The paper was not intended to go into matters of design, except in a very general way. Only the question of theory was involved. For this reason, no detailed design data were considered necessary.

Captain Reed states the generally known fact that the ordinary accepted theories for the centrifugal pump do not even come close to agreeing with observed test results. His explanation is typical of that now given great prominence abroad, to the effect that it is due to the difference between the actual angle of discharge and that obtained from the ordinary diagrams. While the writer does not doubt that there may be some slight difference, it is almost impossible to imagine any such difference as would bring about agreement. The writer suggests that Captain Reed try such a calculation once and see what the difference would have to be. This is only one of a number of straws that have been snatched at in an effort to save the present theory.

Mr. Munk registers his belief in what he terms the "old established blower theory." Contrary to his statement the writer has not denied the existence of fluid pressure, and is perfectly aware that velocity and pressure may and usually do exist independently in a fluid. What the writer did say was that the pressure in the pump-case is not a separate and distinct amount of energy from the velocity which produced it and that pressure is not energy any more than force is work, when applied to an incompressible fluid like water. The writer never claimed the maximum pressure to be $\frac{V_2^2}{2g}$ for centrifugal pumps; all those described in the paper develop heads exceeding that amount. The reason for the amount, $\frac{V_2^2}{2g}$, is that in most of the current texts such a value is given for the head developed at shut-off. In many cases it agrees fairly well with practice, but there are, of course, exceptions.

In replying to Professor Harris it will be convenient to take each paragraph separately. The answer to Item (*a*) is a matter of opinion. Some of the recent writings are after all not so different from those of twenty-five years ago. To be more specific Equation (37) which Professor Harris claims for his own has been in existence even longer.

In Item (*b*) the writer feels that Professor Harris has greatly magnified the difficulties encountered in the ordinary application of Bernoulli's theorem.

Item (*c*) is "hair-splitting" and affects the amount, but not the principle.

Item (*d*) again involves the old question: "Is the head at shut-off $\frac{V_2^2}{g}$, or $\frac{V_2^2}{2g}$?" Professor Harris claims $\frac{V_2^2}{g}$, which opinion is shared by some discussors and denied by others. The writer maintains that no such head, regarded as energy, was ever present nor can it in any manner be utilized.

Item (*g*) is badly confused and the writer cannot see its bearing on the question. Such a discovery would surprise the writer fully as much as it did Professor Harris.

In Item (*h*) the fact as he states it is true, but his conclusions are far from correct as explained under Item (*j*). Neither Equation (37) nor the other data for which he asks, would be of any great help.

As to Item (*i*) the writer did not state that the jet had to have a greater velocity than the paddles although it would have, of course, in the case of radial or forward curved vanes.

In Item (*j*) Professor Harris assumes the rôle of a prophet and speculates on the probable values of the design angles, etc. While not wishing to go into the complete design data the writer can say that not a single guess was correct. Furthermore, the writer has done as Professor Harris suggested, and has tested the pump with forward curved vanes. The efficiency was not below 50%, as Professor Harris claims, but a value well above that. His Equation (37), as given, is the equivalent of Equation (*C*) and does not apply to a centrifugal pump at all. This equation is old and has been commonly used by many writers. Professor Harris gives a method of separating and analyzing the losses in a pump and suggests that it may enlighten designers who are trying to improve the efficiency of their pumps. It is not difficult to understand why designers have not made more use of his method. Professor Harris states that $F = D - Q\,H$ and then that $H_1 = \frac{F}{Q}$, in which, D is the dynamometer input to the pump, F is the energy dissipated in the pump, Q is the discharge, and H is the head developed by the pump. H_1 is said to be the head lost between intake and discharge. This is impossible since in his reasoning he has assumed that all the losses in the pump are of a hydraulic nature. He seems to have completely forgotten that mechanical losses such as bearing friction, stuffing-box friction, and disk friction of the impeller form the bulk of all the losses.

Mr. Gruenberger wonders why only the sum of the three given velocity heads should be used, and further suggests that if another were added the result would be different. The writer knows only three points of control in the pump and hence the three velocity heads. As to the shut-off heads in Figs. 9, 10, and 11, these were obtained by test.

Many of the points in the discussion of Professor Hooper are exceedingly well taken. The head he mentions, registered by a piezometer, measures merely a force and does not represent energy separate and distinct from the velocity which produced it. One is the measure of the other. Centrifugal force is not energy in itself, but it may serve to measure and indicate the presence of kinetic energy.

The discussion by Professor Mavis is of interest as it gives data obtained in various hydraulic laboratories in Europe. The data in Table 1 impress one as being rather too academic. In quite a number of years' experience as a pump designer the writer has never found any such difference in performance, due to a variation in the number of vanes, as these data would imply. He also disagrees with the fundamental assumptions on which the computations were made. A theory based on an infinite number of vanes is not rational and is not at all necessary in order to explain the present theory. It is used on the incorrect assumption that complete guidance of the water could not be obtained with any finite number.

Professor Medici supports the writer's claims as to the fallacies of the theories found in most of the modern texts. Professor A. Stodola is also said to agree with Professor Medici that the head at shut-off is $\frac{V_2^2}{2g}$ and not $\frac{V_2^2}{g}$. This, again, adds to the conflict of opinions expressed in the discussion. The writer does not feel that Fig. 19 in any way explains a rising characteristic accounted for by his new theory. His views on the effect of the vane angle, β_2, on the pump characteristic apply only to an impeller discharging into air and without a case. Under such conditions the question is merely one of simple trigonometry, and any one can verify it for himself. In such cases the results of the angles, β_2, equal to 60° and 120°, would not be at all the same, as claimed by Professor Medici. Construction of the vector diagrams will show this. The criticism of Fig. 16 is based on a misunderstanding. The writer did not claim that the head at shut-off was always $\frac{V_2^2}{2g}$. In fact, he showed that under certain conditions heads much higher than $\frac{V_2^2}{2g}$ were easily obtained. Fig. 16 was for the purpose of showing that the circular-case pumps were capable of greater heads for the same value of V_2 than the ordinary volute pump. The writer does not believe that the various assumptions of vane pressure, recirculation, etc., given by Professor Medici, are sufficient to explain the observed differences. They all involve a loss of energy while accurate tests will readily show that no such loss has taken place.

Professor Blaisdell goes on record as believing the head developed to be $\frac{V_2^2}{g}$ rather than $\frac{V_2^2}{2g}$ and gives an illustration to prove his point. The illustration and his reasoning are both correct, but it is easily perceived that the machine he describes is not a centrifugal pump. The variation in the angle, β_2, emphasized by Professor Blaisdell has been mentioned by others, but a few simple computations will show it to be entirely inadequate to explain errors as large as almost 100 per cent.

Professor O'Brien's discussion is of interest, but does not apply directly to a centrifugal pump. His criticism that the writer neglected to consider the stretching of the wire is not well taken so far as pumps are concerned, because the water is assumed usually to be incompressible.

The discussion of Mr. Donnell contains some unusual statements which the writer is quite unable to follow or accept. For example, he states that,

"The pressure of any part of a body of rotating liquid is not due to its velocity; it would be possible to destroy its velocity and still have the pressure remaining exactly the same as before."

Certainly a liquid cannot rotate without having velocity, and if it does not rotate it has no pressure unless exerted by some other agency.

The writer is indebted to Dr. Timoshenko for a very clear and concise statement of the points of difference. He makes use of two illustrations

to prove that the pressure head and velocity head may be added to give the total head, and his reasoning is correct. He has simply made the old error of assuming that he was describing the usual form of centrifugal pump. In Dr. Timoshenko's illustrative example of the ball moving radially in a groove, (Fig. 34), he calls attention to the increase in the time of action due to the opposing force. The writer does not agree that time of action has anything to do with the work accomplished, since work is merely force times distance and time is not considered. Dr. Timoshenko develops a case in which the total head may very properly be regarded as $\frac{V_2^2}{g}$. He then considers the question of the water flowing radially in the groove opposed by the back pressure of the water in the surrounding case, and concludes that the work done on each pound of water will be exactly the same as before. That this view is not correct can be proved easily by carefully made tests.

Consider an example, as shown in Fig. 45, consisting of a radial piece of pipe so arranged as to revolve about a center at O, with an angular velocity, ω. The outer end of the pipe is plugged and a hole is made in the plug through which water may flow. The pipe is now filled with water and rotated about a center, O, as shown. Assuming a source of supply at atmospheric pressure at the center, water will flow continually through the orifice at the end with a relative velocity, W_2, equal to the peripheral speed, V_2. Thus, the absolute velocity is $\sqrt{2}\ V_2$ and the kinetic energy per pound of water is $\frac{V_2^2}{g}$. This is substantially what Dr. Timoshenko has claimed, and the writer agrees with him in its correctness. It has also been confirmed by tests, as will be shown later. (See page 98, *et seq.*)

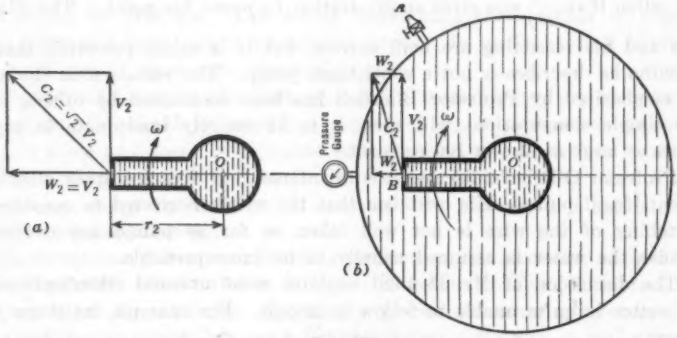

FIG. 45.

Now, assume that the same whirling radial piece of pipe (Fig. 45(a)) is mounted in a concentric circular casing which completely encloses it (Fig. 45(b)). This enclosing case is provided with an outlet through which the discharge may be regulated, as at A. Assume, further, that the entire case and radial piece of pipe are filled with water and that the pressure at the center, O, is atmospheric; also, that the valve at A is closed. The water

in the radial whirling pipe under centrifugal force will try to move out from the center and flow through the orifice at B. However, the space outside is completely filled and, since water is practically incompressible, no flow can take place. Its desire to get out will be communicated to all the particles outside in the case and will appear as a static pressure equal to $\frac{V_2^2}{2g}$ if the water in the case does not rotate with the revolving radial pipe.

A drop of water at Orifice B (Fig. 45(b)) has only a kinetic energy $= \frac{V_2^2}{2g}$ per lb., and that is the only work done on it by the motor which drives the revolving pipe. This work was done during the acceleration period and after it is once up to speed no further work is required except to overcome the various mechanical losses. It is easily seen that the velocity and pressure are (as was stated) "part and parcel" of each other. Either may be regarded as a measure of the energy, but not both. The pressure exists because of the velocity and depends on it. Were there no velocity there would be no pressure (Mr. Donnell to the contrary notwithstanding.)

Now, if the valve at A is opened a little some water will flow out. The pressure in the case will be reduced, and since the back pressure which, before the valve was opened, was just equal and opposite to the head developed by centrifugal force tending to produce flow, is now reduced, water will flow out through Orifice B to replace that flowing at Orifice A. The relative velocity at Orifice B will be due to the difference between the centrifugal head, $\frac{V_2^2}{2g}$, and the back pressure in the case. It will be small if the pressure difference is small and will approach a value, $W_2 = V_2$, if the back pressure is atmospheric. This means that, for all cases of back pressure in this casing, the relative velocity, W_2, will be less than V_2, as shown in Fig. 45(b).

The work done on each pound of water passing through this machine is measured solely by the amount of kinetic energy in the absolute velocity of discharge, C_2. (This is not in agreement with Dr. Timoshenko.) The energy of the water is zero when it enters, and $\frac{C_2^2}{2g}$ when it leaves, the orifice, B; hence the work done is $\frac{C_2^2}{2g}$ for each pound. Fig. 46 (a) shows how the back pressure in the case affects the absolute velocity, C_2, and hence the work done. C_2 (maximum) $= \sqrt{2}\, V_2$ is obtained with zero pressure in the case, and C_2 (minimum) $= \frac{V_2^2}{2g}$ occurs when practically no flow takes place and the pressure in the case is $\frac{V_2^2}{2g}$. This explains why the shut-off head is only $\frac{V_2^2}{2_1 g}$ and not $\frac{V_2^2}{g}$.

The situation may be compared to that shown in Fig. 46(*b*) and (*c*). If the water level is the same in each tank (Fig. 46(*b*)), no flow will take place and practically no work would be required to get a particle of water from one tank to the other. However, in Fig. 46(*c*) the levels are not the same and, hence, flow will occur due to the head, h, as shown. In case a water motor or pump were located in the connecting pipe (Fig. 46(*c*)), power would be obtained from the head, h, or work would have to be done against only that head to get water from the lower to the higher tank. This fact is important. In each case the work done can be calculated by determining the relative velocity, W_2, due to the pressure difference and combining this with the peripheral velocity, V_2, as a vector to obtain the absolute velocity, C_2.

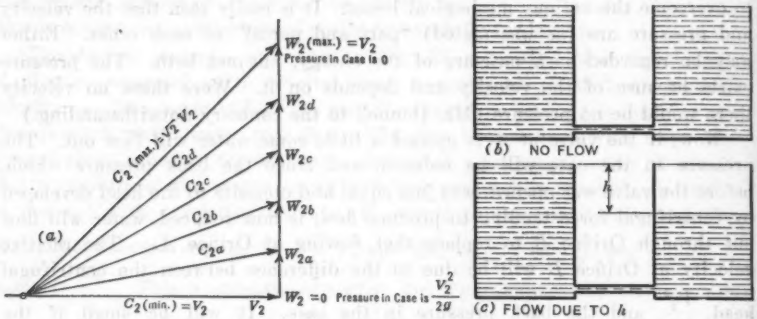

FIG. 46.—EFFECT OF BACK PRESSURE IN FIG. 45(*b*) ON THE RELATIVE AND ABSOLUTE VELOCITIES.

Dr. Timoshenko would say that the work done is always the same and is equal to $\frac{V_2^2}{g}$ no matter what the pressure and hence no matter what the absolute velocity. His idea is that the work done is divided into two parts one to produce the absolute velocity, C_2, and the other to get the water out into the case against the back pressure. Fig. 46(*c*) should make clear this fallacy because, if there is a level difference as shown, flow will take place without any additional expenditure of work. It is difficult to understand how Dr. Timoshenko's claim could be sound mechanically even if it were hydraulically correct for the reason that if the water is to do work against the back pressure in the case the direction of this force must be radial and balanced all around. Since such a force could have no tangential component, it is difficult to understand how it could possibly affect the turning of the wheel which is the only way power could be consumed.

The problem was so clear cut that it was decided to try an experiment which would settle the question absolutely. For this purpose an apparatus similar to that of Fig. 43(*b*) was made up, as shown in Fig. 47. It consisted of a piece of 2-in. pipe to which were connected the two radial arms shown. The pipe passed through stuffing-boxes in each side. It also continued on one

side into an open tank containing water and on the other was connected to a small electric dynamometer. The supply of water to the open tank was arranged so that the water level was maintained a few inches above the opening through which the 2-in. pipe entered. The box was air-tight and was provided with a discharge valve in the bottom and a pipe leading to the compressed air tank at the top. A gauge was also provided to record the pressure in the box.

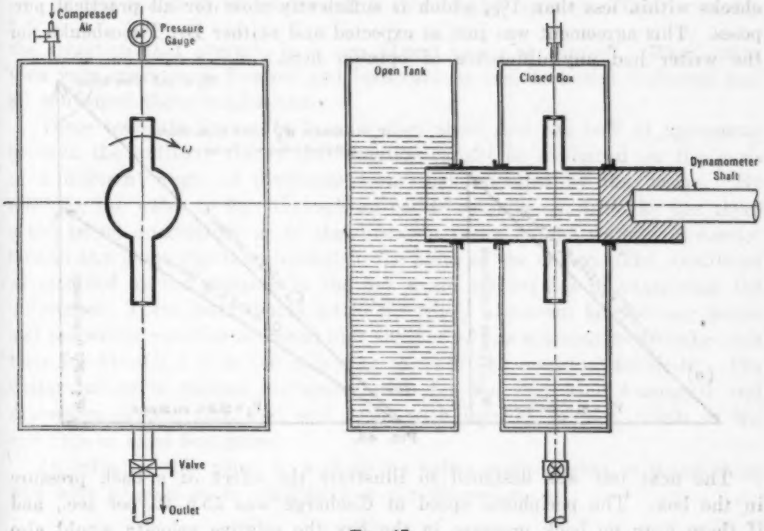

FIG. 47.

It was possible to maintain any desired constant pressure in the box by regulation of the air inlet valve on top and the water discharge valve below. The dynamometer was allowed to run without water in the rotating arms and the "no-load" reading was taken. This consisted of a slight amount of stuffing-box friction and windage. Then water was allowed to flow into the 2-in. pipe and out through the arms. The power required to drive the machine was then observed and corrected for no-load losses. The pressure (if any) was maintained constant during each test. The quantity of water discharged was determined by weighing on an accurately calibrated scale. In this way tests could be made either with atmospheric pressure in the box or higher as desired.

The first tests were made at atmospheric pressure. The final results checked almost exactly with what would be expected under such conditions. The peripheral speed of the end of the rotating arms was 81 ft. per sec. This was found by observing the radius and the rotative speed. According to theory the relative velocity, (W_2), of the water discharged from the orifice should be the same as V_2, or 81 ft. per sec. The absolute velocity would be

$\sqrt{2}\ V_2 = \sqrt{2} \times 81 = 114.5$ ft. per sec., as shown in Fig. 48 (*a*). The head per pound of water would be,

$$\frac{C_2^2}{2g} = \frac{(\sqrt{2}\ V_2)^2}{2g} = \frac{V_2^2}{g} = \frac{81^2}{32.2} = 204 \text{ ft.}$$

The quantity of water discharged was 22.45 gal. per min. The power required to raise 22.45 gal. per min. to a height of 204 ft., would be 1.156 h.p. The power actually measured by the dynamometer was 1.142 h.p. This checks within less than 1%, which is sufficiently close for all practical purposes. This agreement was just as expected and neither Dr. Timoshenko nor the writer had any difference of opinion here.

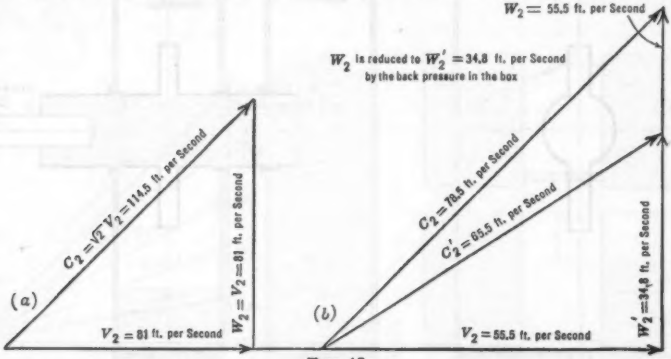

FIG. 48.

The next test was designed to illustrate the effect of a back pressure in the box. The peripheral speed at discharge was 55.5 ft. per sec., and if there were no back pressure in the box the relative velocity would also be 55.5 ft. per sec., and the absolute velocity 78.5 ft. per sec., as shown in Fig. 48(*b*). However, a pressure of 12.6 lb. per sq. in. was maintained in the box during this run. The head due to centrifugal force tending to produce flow is $\frac{V_2^2}{2g}$, or $\frac{55.5^2}{64.4} = 47.9$ ft. This is opposed by a back pressure in the box of 12.6 lb., which is equal to a head of 29.1 ft. The net head producing flow through the orifice is the difference between these two, or 18.8 ft. This head will produce a relative velocity, $W_2' = 34.8$ ft. per sec. If this relative velocity, W_2', is laid off at right angles to V_2 in Fig. 48 (*b*), an absolute velocity, $C_2' = 65.6$ ft. per sec., is obtained. The kinetic energy represented by this is equivalent to a head of $\frac{65.5^2}{64.4} = 66.5$ ft. The quantity of water discharged was found to be 8.88 gal. per min. The power required to lift 8.88 gal. per min. to a height of 66.5 ft., would be 0.1495 h.p. The power shown by the dynamometer was 0.1475 h.p. This agrees within practically 1% and proves absolutely that no work is done against the pressure in the box, as was claimed by Dr. Timoshenko. If he were correct, the power required would have been 0.2150 h.p. instead of 0.1495 h.p. as measured.

In other words, it has been shown by actual tests that the head is not $\frac{V_2^2}{g}$ in such cases, but is a quantity less than that, which may be computed. If the pressure head in the box was increased until it was nearly equal to $\frac{V_2^2}{2g}$, the flow would be reduced to almost zero and the head would approach $\frac{V_2^2}{2g}$, as was shown in the paper and is now verified by test. This is exactly the situation that prevails in a centrifugal pump at shut-off. Many other tests were made with forward and backward as well as radial discharge and all confirmed these conclusions.

These tests are important because they show that the lack of agreement between the ordinary theory and practice cannot be explained on the basis of a different angle of discharge from that intended by the design. By allowing the water to flow through an orifice radially out into the box there could be no uncertainty as to the direction taken by the water. It cannot flow in any other way than normal to the plane of the orifice. The conditions as modified by the pressure in the box alone are capable of explaining the difference. These tests should settle the whole argument beyond any doubt and the writer would recommend that any one who is not convinced make such tests for himself. It is the only way to settle the question definitely. The writer wishes to express his appreciation for the very kind comments and discussion of Mr. Cox, and will await with interest further details of his new type of axial flow pump.

In bringing this paper to a close the writer again wishes to thank those who were so good as to contribute to the discussion.

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Paper No. 1698

THE GREAT YANGTZE BAR*

By A. V. Hugo von Heidenstam,† M. Am. Soc. C. E.

With Discussion by Messrs. A. W. Robinson, and R. L. Vaughn.

Synopsis

The object of this paper is to describe the engineering investigations of the Yangtze Estuary for purposes of improving navigation to and from the Port of Shanghai, China. Since the opening of the river to foreign trade in 1842, the volume of shipping entering it has grown enormously, now (1927) amounting to about 16 000 000 British registered net tons per year. Within recent years the draft of ships has increased to a figure at which navigation over the bars of the Yangtze has become difficult. The interests of the Yangtze ports in general and of Shanghai in particular call for improvements.

The engineering problem is one to daunt the most courageous. The bars are many miles in length and the channels 2 or 3 miles wide. The depth in the main channel is 19 ft. at ordinary spring low water, 27 ft. at neap high water, and 31½ ft. at ordinary spring high water. The material is mud, and the bars are bordered by enormous shoals of similar mud, some of which shift and re-form constantly. The water is always heavily charged with silt and while there is a fresh-water discharge perhaps exceeding in maximum value that of any other river in world, its effects are practically neutralized in the estuary by a prodigious tidal flow.

The changes caused by tidal forces in the course of a few days are sometimes greater than from the largest conceivable excavating machinery in many months. The site is fully open to the sea. The main channels, separated by shoals, have cross-sections of about 1 000 000 sq. ft., widths of 3 to 4 miles, and depths in their shallowest sections over the bars of about 19 ft. at ordinary spring low water. There is no solid foundation for training works and their magnitudes would be appalling from a financial point of

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view. Under all circumstances a record of the data collected cannot but be of general interest.

A summary is given of the investigations made by the writer, as Engineer-in-Chief, for the Whangpoo Conservancy Board, since 1914, culminating in a special investigation, from 1919 to 1921, as to the possibilities of improving the approaches to, and the facilities in, Shanghai Harbor. This investigation was concluded by the convening of an International Committee of Consulting Engineers, consisting of seven river and harbor experts, who reported at the end of 1921. The principal recommendations made by this Committee included the dredging of the South Channel and the construction of a system of public wharves in the Whangpoo River. This report is still (1927) under consideration by the Central Government.

Some of the difficulties involved in the dredging proposal are noted. An Appendix is also attached, dealing with the principal technical data collected.

Introduction

The problem of improving the sea approaches to Shanghai as here described is most difficult. Shanghai is situated on the Whangpoo River, about 15 miles from its mouth, the river itself forming the harbor (Fig. 1). The Whangpoo enters the estuary of the Yangtze about 30 miles above the main bar of that river, the "Fairy Flats". The depth on the crest of that bar at extraordinary low water is only about 16 or 17 ft.; at ordinary spring low water, 19 ft.; at neap high water, 26 to 28 ft.; and at ordinary spring high water, 30 to 32 ft. There are two other entrances to the Yangtze, but one of these is quite beyond improvement, while the other is similar to the Fairy Flats and more indirect.

The shipping visiting Shanghai has increased to such a volume that, reckoned by tonnage, this city is among the first six ports of the world. Moreover, the drafts of the ships are steadily increasing. Many of these ships have drafts of about 30 ft., and an increase in draft is certain within a few years. A vessel of 30-ft. draft is delayed only occasionally, but those of larger draft would have great difficulty. The deepening of one of the bars, therefore, is indispensable to the future welfare of the Port of Shanghai.

The magnitude of the operation is in itself appalling. Training works of accepted types and of an extent proportionate to the size of the bar might easily cost $50 000 000 (gold). Even so they would not be of such stability as to assure undeniable success. Dredging is only conceivable if the very largest and most efficient modern machines are to be used. The mere surveying and marking out of the site of the works is a very large operation. All these circumstances enhance the technical interest of the investigations already made and the professional opinions given.

History and Record of Reports

In 1876, Messrs. J. de Rijke and G. A. Escher, two Dutch civil engineers employed by the Japanese Government, were requested by the Shanghai

Consular Body to report on the Inner Bar of the Whangpoo River (which has since been eliminated by the works of the Whangpoo Conservancy Board). A supplement to their report included an "Essay on the Fairway from and to Shanghai through the Estuary of the Yangtze."

At that time it appeared from a comparison of the British Admiralty charts issued since 1842 that the South Channel (Fig. 1) was becoming shallow. More recent maps show that this tendency, if at all a fact, ceased subsequently. The authors of this 1878 report believed that the North Channel (Fig. 1) was growing at the expense of the South Channel and that by closing various small crossings and skilfully cultivating a channel between the shoals opposite the mouth of the Whangpoo, a good passage to the North Channel could be developed.

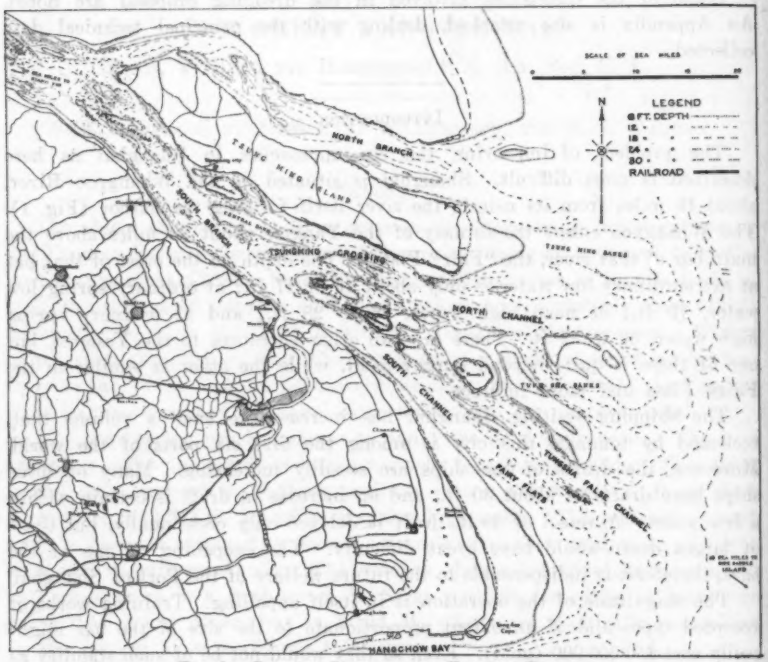

FIG. 1.—GENERAL PLAN OF THE YANGTZE ESTUARY.

It should be noted that the outer bar in the North Channel is usually deeper than that in the South Channel, the real obstruction being an upper bar (Tsungming Crossing). They had in mind a low-water draft of only 15 ft. In their opinion mechanical dredging was impracticable:

"* * * even for assisting only it must prove too expensive in so extensive an estuary. The dredgers, too, as well as the mud scows, would require to be built in a specially costly manner to fit them for navigating rough and dangerous water; but even if they were so constructed, still the number of off days would be very great."

Again,

"Everybody will also understand that any attempt to dredge the estuary of the Yangtze would be throwing money away, and that a single storm would destroy many months' work."

This very definite opinion given in 1876 is probably still true with respect to bucket dredges (which are distinctly suggested by the reference to scows), in spite of the great increase in capacity of dredges. It may not be true, however, of modern suction dredges some of which have a very large output and, by the use of hoppers or pipe lines, become independent of a scow service.

The report has interest as giving a word of warning to Shanghai, but has only a limited bearing on the present problem.

Whangpoo Conservancy Board.—In 1906 the Whangpoo Conservancy Board commenced operations in the Whangpoo with Mr. de Rijke as Engineer. In 1910 certain works near the mouth of the Whangpoo were almost completed, but operations had to be stopped owing to lack of funds. At the end of 1910 Mr. de Rijke was succeeded by the writer, and a re-organization was effected. In the writer's "Project for the Further Regulation of the Whangpoo" of 1911, which work was successfully completed in 1922, attention was drawn to the necessity of watching the behavior of the Yangtze. As part of that project (1912-22), a hydrographic service was established on the Whangpoo, including automatic tide gauges and regular velocity, discharge, and silt measurements. Incidentally, these provided much information as to the Yangtze, as the Whangpoo is really not much more than a tidal "swashway", or large creek, dominated by the Yangtze.

In 1914 the writer recommended that the Board make a special investigation of the Yangtze Estuary, to indicate whether any changes were occurring which might affect the welfare of the Whangpoo, and, at the same time, to make quite clear the condition of the sea approaches to Shanghai. This work was carried on in co-operation with Coast Inspector's Office of the Chinese Maritime Customs, and much hydrological information, in the form of charts (including areas previously unmapped), and tidal, current, and silt data, were accumulated. No specific attempt was made to formulate plans for regulating the estuary, but a working model was built in the hope of reproducing the currents and ascertaining what changes would arise from certain classes of work. Scale difficulties, however, prevented the success of this experiment.

As a result of this investigation, and in order to consider future needs, it was decided to take up the general question of possible improvement. In 1917, by request of the Board, the Vattenbyggnadsbyran, Consulting Engineers, of Stockholm, Sweden, and the writer, submitted a "Report on the Future Development of Shanghai Harbour", which analyzes Shanghai's position in regard to trade routes and includes a definite technical consideration of the approaches. A "Memorandum on a 'Port de vitesse' for Shanghai on, or Canal to, the Hangchow Bay" (see Fig. 1) was appended.

Shanghai Harbor Investigation.—A great deal of interest was created by the publication of this report (of which the definite recommendation was a further and full inquiry), and, in 1919, the Board authorized and commenced

a "Shanghai Harbor Investigation" which was to include as complete a survey as possible and, in addition, projects for improvement of the approach and the construction of harbor works. The work was carried on in three sections under the writer's direction, as follows:

Section I.—Surveys of estuary, etc., in charge of E. C. Stocker, M. Am. Soc. C. E.
Section II.—Harbor designs, in charge of Mr. C. W. Simon.
Section III.—General studies and secretarial, in charge of Mr. H. Chatley.

The delicacy of the approach problem was felt to be such that in addition to the Shanghai investigation two consulting engineers were retained, Mr. C. A. Jolles, of Arnhem, Holland, and Vattenbyggnads-Byran (The Hydraulic Engineering Bureau), of Stockholm.

It was part of the duties of the latter firm to consider also the desirability of making a new approach by means of Hangchow Bay (thus avoiding the present bars altogether). Similarly, Section I was to make (in co-operation with the Chinese Maritime Customs) a full survey of Hangchow Bay (Fig. 1). This survey revealed that the depths in this Bay were just sufficient for the largest drafts likely to visit Shanghai. Thus, the advisability of adopting this method of avoiding the obviously great difficulties of improving the Yangtze depended on two questions:

(*a*) Whether Hangchow Bay was likely to maintain its present depths, or, if not, whether a deep channel could be artificially maintained? and,
(*b*) Whether constructing a first-class ship canal or, as an alternative, a new coast "port de vitesse", with good water connections to Shanghai, was economically practicable?

The Vattenbyggnads-Byran expressed the very definite opinion that Hangchow Bay was slowly but steadily silting up. The survey and comparison with other charts partly confirmed this opinion. As the Bay is quite open and without continuous current channels the possibility of deepening it artificially if required in the future seemed very doubtful.

In the matter of the connection with Shanghai, a ship canal with locks of the Panama Canal type to exclude salt, silt, and tide, appeared prohibitively expensive. The "port de vitesse" in itself was also a matter of great cost and would have a very uncertain relation to Shanghai's peculiar economic welfare.

The Vattenbyggnads-Byran expressed the opinion that the "idea of reaching Shanghai *via* the Hangchow Bay should be definitely given up."

Mr. Jolles, with his long experience on Dutch waterways, was asked to report on the estuary problem only, and expressed the opinion that the bar was due largely to coagulation of the silt by saline flood currents; that the South Channel was the only one for which regulation should be attempted; and that training works would be prohibitively expensive and uncertain of success, but that dredging might be successful. He estimated that 25 000 000 cu. yd. of material might be involved. No "streaming away"* need be anticipated as the silt precipitation will not undergo any change. Drag suction dredges might be

* Removal of mud in excess of the quantity dredged by reason of the current acting on the disturbed bottom.

the only suitable type. He finally suggested dredging work in the channel connecting the "Tungsha Channel" to the South Channel (southwest of House Island, Fig. 1) and in the Tungsha Channel itself, to a depth of 25 ft. below low low water, involving about 12 500 000 cu. yd. of dredging and 900 000 sq. yd. of mattress protection. The cost would be about 22 000 000 Taels (say, $15 000 000); the permanence would be uncertain.

The Tungsha Channel (Fig. 1) is a relatively recent development. The South Channel was formerly wider and then was split by the shoal now termed the Tungsha Spit (Fig. 2). The Tungsha Channel is really part of the former South Channel and for some years it was expected to become eventually the navigation route, usurping the place of the narrowed course of the South Channel. In recent years, however, it has shown no tendency to increase in depth and the bar at its head (which Mr. Jolles proposed to dredge) has shallowed.

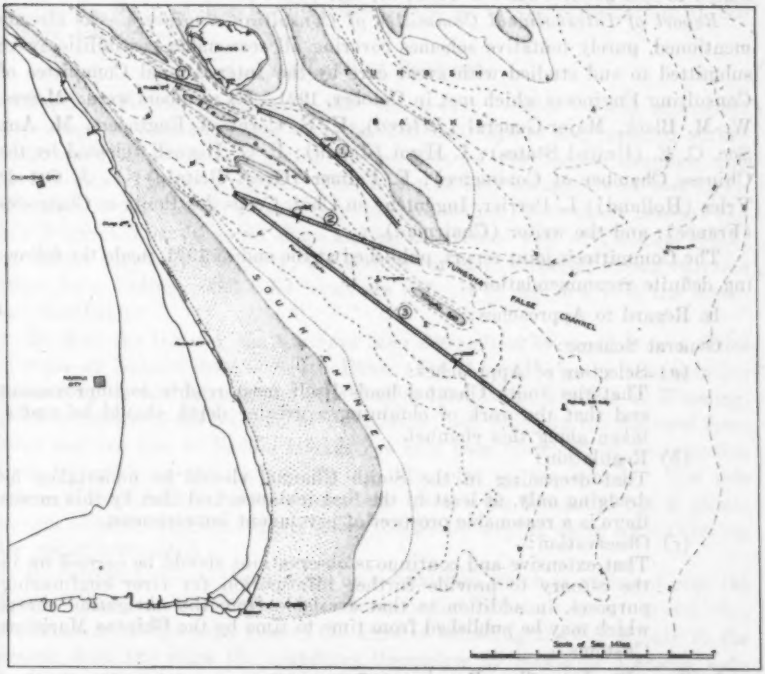

FIG. 2.—THREE LAYOUTS FOR DREDGING PROJECTS IN SOUTH CHANNEL, YANGTZE RIVER.

The Vattenbyggnads-Byran in its report made an elaborate but rather academic study of the shoal movements and concluded that the only permanent tendency is for the deepest channel to swing southward. It held, therefore, that the present South Channel is the most stable of all and definitely recommended dredging along the axis of that channel.

On the basis of the local investigation and studies, the writer, prior to the arrival in Shanghai of the International Committee of Consulting Engineers in October, 1921, prepared a general report on the entire problem (the harbor itself and its approaches) for the use of the Committee, which included plans and estimates for various solutions.

In regard to the estuary, it put forward three alternative dredging projects in the South Channel (Fig. 2):

(1) A cut, 11 miles long, along the thalweg of the Tungsha Channel.
(2) A direct diagonal cut, 9 miles long, between the two channels.
(3) A direct cut, 20 miles long, along the axis of the South Channel.

The net volumes of dredging in the three cases were, respectively: (1) 13 400 000 cu. yd.; (2) 12 500 000 cu. yd.; and (3) 22 000 000 cu. yd.

Suitable harbor works, survey, and dredging craft were included, with five years' operation on whichever of the three layouts might be preferable, estimated to cost about 8 325 000 Taels, or, say, $6 000 000.

Report of International Committee of Consulting Engineers.—As already mentioned, purely tentative schemes covering all reasonable possibilities were submitted to and studied with great care by the International Committee of Consulting Engineers which met in October, 1921. Its members were: Messrs. W. M. Black, Major-General (*Retired*), U. S. Corps of Engineers, M. Am. Soc. C. E. (United States); I. Hiroi (Japan); P. G. Hornell (elected by the Chinese Chamber of Commerce); F. Palmer (Great Britain); P. J. Ott de Vries (Holland); L. Perrier, Ingénieur en Chef, Corps des Ponts et Chaussées (France); and the writer (Chairman).

The Committee's joint report, published at the end of 1921, made the following definite recommendations:

In Regard to Approaches:

General Scheme:

(*a*) Selection of Approaches:
That the South Channel lends itself most readily to improvement and that the work of obtaining a greater depth should be undertaken along this channel.

(*b*) Regulation:
That deepening in the South Channel should be undertaken by dredging only, at least in the first instance, and that by this means there is a reasonable prospect of permanent improvement.

(*c*) Observation:
That extensive and continuous observations should be carried on in the estuary to provide further information for river engineering purposes, in addition to that available from the navigation charts which may be published from time to time by the Chinese Maritime Customs.

In Regard to Immediate Requirements:

(*a*) To carry out forthwith the river engineering observations indicated under the "General Scheme."

(*b*) To make and maintain a channel through the Fairy Flats, developing it gradually foot by foot, to as great a depth and width as required and possible, and to provide as soon as feasible a channel of 600 ft. bottom width for the passage of ships drawing 33 ft. at ordinary neap high water.

The estimate for the work, under "Immediate Requirements", for the improvement of approaches, was as follows:

Capital Expenditure:		
Survey plant	Shanghai Taels	200 000
Dredging plant	" "	4 200 000
Incidental plant	" "	125 000
Total	Shanghai Taels	4 525 000
Annual Expenditure:		
Survey	Shanghai Taels	100 000
Dredging	" "	1 000 000
Total	Shanghai Taels	1 100 000

This plan is still (1927) under consideration by the Central Government and its adoption involves an agreement between that Government and the Diplomatic Body of Foreign Representatives as to the constitution of the Harbor Authority.*

The Present Project†

Survey.—It will be observed that the actual location of the attempted deepening has been left open, with the idea that further and repeated survey work would throw more light on the position of the deepest channels and the stability of the bed masses. This is not as easy as appears at first sight. The river is too wide to permit very exact re-section; permanent beacons can only be erected on the south shore and on the islands west of Tungsha Bank; the depths vary only gradually; the bottom varies in softness; strong currents affect the soundings; and tidal levels are difficult to compute with the necessary exactitude.

To date, the tides on the bar have been determined by reading on a series of poles at Nanhui on the South Bank (Fig. 1) and also by interpolation between an automatic gauge at Side Saddle Islands and another at Woosung, or by temporary poles on some of the islands; but owing to the distance from shore the tide pole at Saddle Islands has only been "leveled" by computation from water slopes—similarly with the tide poles on the islands. The tide poles at Nanhui have been connected by levels with Woosung, but it is not certain whether the high waters across the gently sloping shore out to the bar are really horizontal.

Soundings have also been taken on a lightship, which is anchored over the Fairy Flats and swings over a moderately flat bottom. It is obvious that the accuracy of the tidal rise at the time of sounding the bar is only to the nearest foot, and since the soundings themselves by reason of currents, softness of mud, etc., are probably uncertain by a similar amount, the actual levels of the bottom (Figs. 3, 4, and 5) are in doubt by as much as perhaps 2 ft.

* No criticisms have yet been made by outside parties on the estuary project, except a paper by Mr. Canter Cremers, in the September, 1923, issue of *The Dock and Harbour Authority* (London), in which he expresses the fear that the inward running flood currents will neutralize the dredging by reason of their excess of force on the bed over the ebb currents, due to salinity.

† As outlined by the International Committee of Consulting Engineers, 1921.

Over a large area of the bar the computed values differ only by 1 or 2 ft. in distances of ½ mile. The top, therefore, is practically level, and it is a question whether there is any definite thalweg across it. Longitudinal profiles of the South and North Channels are shown in Figs. 3 and 4. Fig. 5 gives a cross-profile of the estuary at the shallow part (Fairy Flats). Elevations are referred to Woosung horizontal zero (W. H. Z.), an imaginary horizontal plane through the lowest low water at the mouth of the Whangpoo.

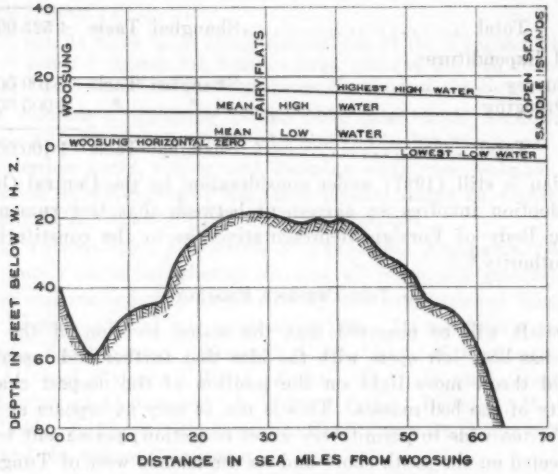

FIG. 3.—LONGITUDINAL PROFILE OF FAIRY FLATS.

A summary of the tide on the Fairy Flats is given in Table 1. At spring tides the tidal currents may be as strong as 6 knots. The silt content may rise to 2 000 parts per million by weight and the salinity of flood currents may be more than one-half that of true sea water.

TABLE 1.—STANDARD WATER LEVELS AT FAIRY FLATS (NANHUI BEACON).

	Water level referred to W. H. Z., in feet.	Rise, in feet.	Depth on crest, in feet.	SEASONAL CORRECTION TO BE APPLIED FOR:	
				January.	July.
Highest high water	18.0	21.8	37.8	.....	.....
Standard spring high water	11.8	15.6	31.6	−0.33	+1.35
Mean high water	9.6	13.4	29.4	−0.62	+1.24
Highest low water	8.6	12.4	28.4	.....	.....
Standard neap high water	7.0	10.8	26.8	−0.10	+1.27
Mean water level	5.4	9.2	25.2	.....	.....
Standard neap low water	3.6	7.4	23.4	−1.22	+0.39
Mean low water	0.8	4.6	20.6	−0.80	+0.81
Lowest high water	0.2	4.0	20.0	.....	.....
Standard spring low water	− 0.0	3.8	19.0	−0.28	+0.28
Lowest low water	− 3.8	0.0	16.0	.....	.....

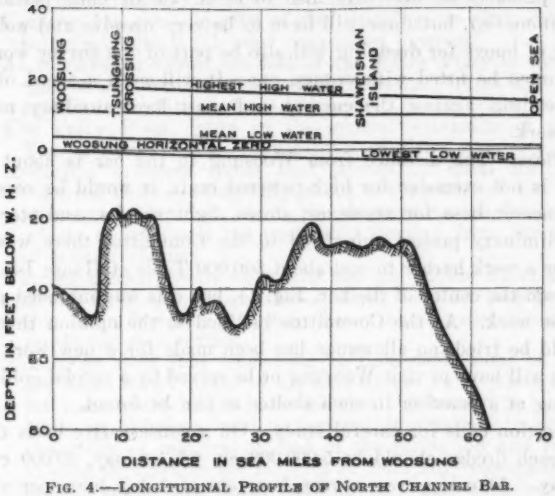

FIG. 4.—LONGITUDINAL PROFILE OF NORTH CHANNEL BAR.

The purchase of a good sea-going survey ship with auxiliary launches, etc., is included in the project. This vessel could repeat soundings with a view to a more rapid determination of progressive changes. By the establishment of numerous temporary tidal stations, it could obtain slightly more precise measurements; but the difficulties are quite serious—the removal or accretion of even 1 000 000 yd. of mud might take place unnoticed.

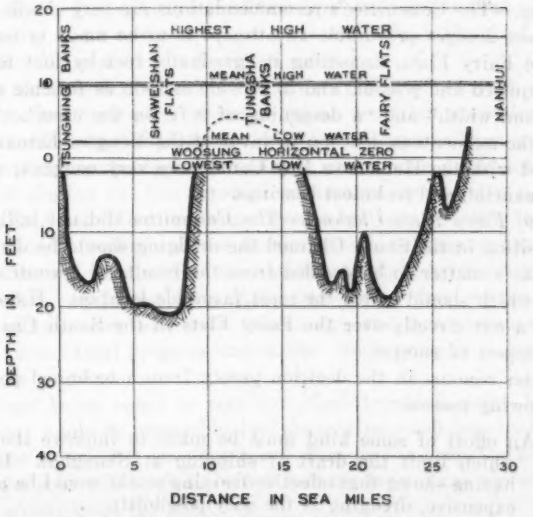

FIG. 5.—TRANSVERSE SECTION OF THE TWO BARS.

It will probably be necessary also to erect one or more permanent tide gauges (automatic), but these will have to be very massive and well founded. The laying of buoys for dredging will also be part of the survey work, so that the vessel must be fitted with cranes, etc. It will need a speed of 10 knots to make headway against the current and must have auxiliary motor craft for shoal work.

Work Base.—The distance from Woosung to the bar is about 30 miles. While this is not excessive for high-powered craft, it would be very desirable to have a nearer base for residence, stores, light repairs, and storm shelter. In the preliminary project submitted to the Committee there was included a design for a work harbor to cost about 500 000 Taels at House Island (about 10 miles from the center of the bar, Fig. 1), but this was intended principally for mattress work. As the Committee inclined to the opinion that dredging alone should be tried, no allowance has been made for a new work base, and the dredges will have to visit Woosung or be served by a special collier capable of bunkering at a mooring in such shelter as can be found.

This question calls for careful study. On a conservative basis the annual output of each dredge should be 5 000 000 cu. yd., or, say, 20 000 cu. yd. per working day. Allowing even a very low rate of 1 h.p.-hour per yd. shifted and 3 lb. of coal per h.p.-hour (includes getting up steam, etc.), the daily coal supply will be at least 30 tons, and more probably 50 tons. The largest drag suction dredge familiar to the writer is the *New Orleans* (Frühling), which has a 3 000-yd. hopper and 300-ton bunkers. For reasons of draft a 2 000-yd. hopper and, say, 200-ton bunkers will probably be the limit for this work, so that each dredge will have to coal about once a week. Oil fuel might be considered.

Dredging.—The Committee's recommendations are very simple. By means of the largest dredges procurable an attempt is to be made to cut a channel through the Fairy Flats, deepening it "gradually foot by foot to as great a width as required and possible and to provide as soon as feasible a channel of 600 ft. bottom width" and "a deepening of 9 ft. on the crest". The reasons leading to the momentous decision in favor of the Yangtze Estuary Approach as compared with the Hangchow Bay Canal were very complex, with various political, financial, and technical bearings.

Choice of Fairy Flats Channel.—The Committee did not indicate exactly in what position in the South Channel the dredging should be done, believing that this was a matter to be decided from the results of a continuous survey of the bar, which should reveal the most favorable location. However, it suggested that a cut directly over the Fairy Flats in the South Channel offered the best prospect of success.

The writer concurs in the decision purely from a technical point of view, for the following reasons:

(1) An effort of some kind must be made to improve the approaches which limit the draft of shipping at Shanghai. Investigation having shown that effective training works would be prohibitively expensive, dredging is the only possibility.

(2) The center line of the South Channel is the most direct navigable course for the majority of the shipping, so that, other things being equal, it is the most convenient location.
(3) As to physical shape, the bed of the Fairy Flats is simpler than that of any other passage, and, therefore, offers the best topographical conditions for improvement.
(4) The Fairy Flats, lying in the general extension of the Lower Yangtze, offers better probabilities of direct scour by the ebb.
(5) Historical evidence shows that permanent shoal formation has occurred more continuously about the mouths of the other two openings rather than about the entrance to the South Channel, which, therefore, is the preferable location from the standpoint of secular change.
(6) Current measurements show that the ebb on the Fairy Flats is as strong as, if not stronger than, that in the other passages. Hence, this site offers the prospect of maximum natural scour and of minimum re-accretion.
(7) Soundings show that from time to time depressions of large area but small depth occur on the axis of the Fairy Flats and persist for some years. This indicates that the bottom is in a state of labile equilibrium, with occasional inclination to scour. The quantities of material shifted in the formation and re-accretion of such depressions are far greater than any reasonable dredging project could cope with, but the fact that such variations of depth do occur suggests that a continuously applied effort on a small area along the line of the strongest currents may co-operate with the scouring tendency of the ebb currents to produce a permanent depression.
(8) These considerations taken conjointly give a reasonable hope that an artificial deepening over a relatively minute fraction of the width of the Fairy Flats would have such a measure of stability as to warrant the attempt to create and maintain it.

Proposed Method of Dredging.—There are two outstanding difficulties: (*a*) The quantity to be removed, which is in the neighborhood of 20 000 000 cu. yd.; and (*b*) the re-accretion that may occur.

There can be little doubt that the slow excavation of a trench to the full depth would form a dead-water pocket in which accretion would be extremely rapid, and a method of dredging that provides a long small cut seems much to be preferred. Since the degree of stability of the sides of the excavation is small, a broad, shallow cut has apparent advantages over a narrow, deep one. On the Taku Bar, near Tientsin, some success has been found with the latter form. So far as the writer is aware the drag suction dredge is the only type that gives the desired results. Ordinary suction dredges (with rotary cutters), or bucket dredges, would not serve the purpose since they cannot work well unless they make a deep and wide lateral cut, the mere volume of which makes rapid longitudinal progress impossible. Such progress is indispensable in order to encourage scour.

Other things being equal, a pipe-line discharge to a point beyond the dredged channel would be preferable, but the marine conditions and the speed required appear to prohibit it. Overboard discharge is open to the objection that part of the mud will be re-deposited over the area of the cut and thereby partly if not wholly undo the work.

The marine conditions suggest that a self-contained hopper dredge will be enabled to work more continuously than a hopper barge arrangement. The decision, therefore, seems to rest exclusively with the drag-suction, self-propelled, hopper-type dredge.

The desired cut increases in length as it deepens, to about 25 miles, but if the shorter shallow depression maintains itself, the stability should increase with the depth, so that the practical question will be rather one of rapidly channeling the bar over a length of, say, 10 miles. If the dredging speed is 3 knots (over the bottom) this means that one through cut might be made in a tidal period (allowing half the time for dumping). If an appreciable result is to be produced in 5 years the annual dredging capacity should be at least 5 000 000 cu. yd., or, say, 2 000 effective cu. yd. per hour (for 2 500 working hours); allowing only one-third the working time for actual dredging, this means a continuous capacity of 6 000 cu. yd. per hour, which would require the largest and most efficient machines yet built. Failure to achieve an output of this size will greatly reduce the possibilities of success unless the natural scour proves very favorable.

The brief details given show the extraordinary uncertainty and difficulty of the problem. Its magnitude appeals to the imagination of every engineer. The Appendix gives further data relating to the estuary.

APPENDIX

General Data

Historical Changes.—Owing to the inaccuracy of early maps there are no precise records of the undoubtedly great changes that have occurred in historic times.

Fig. 6 shows a series of rough maps compiled from old data, which clearly indicate the superior stability of the present South Channel, and the progressive deterioration of the North Branch. The various surveys made since the middle of the Nineteenth Century, the accuracy of which has steadily increased, do not give any distinct evidence as to systematic change in the two main bars; accretion seems to have occurred principally in the regions of the North Branch and Yangtze Cape. Erosion appears to be a purely local phenomenon initiated at concaves.

Tides.—The whole estuary is subject to tidal action. Tidal observations have been made for prolonged periods at Side Saddles (Fig. 7), an island at the extreme north of the Chusan Archipelago in the open sea just off the mouth; at Woosung Forts (Fig. 8), at the entrance to the Whangpoo; and at Kiangyin (Fig. 9), the head of the estuary. Other places have been observed for shorter periods. The annual change of "stage" in the river reduces the tides in the upper part of the Estuary, as shown in Fig. 9. At Kiangyin the mean level has a fortnightly swing.

Tidal Currents.—The currents are quite strong. Maximum flood and ebb velocities of more than 6 ft. per sec. have been observed at a point about 20 miles above Woosung; and on the bar the maximum filament flood velocities at spring tides may be 10 ft. per sec. Outside the estuary there is a distinct clockwise rotary effect and even in the lower parts of the estuary (for example, on the Fairy Flats) the tidal currents move in long flat loops so that there is a transverse (northward) current at slack water.

The non-tidal gradients in the estuary are very small (less than 1 ft. in 10 sea-miles), but the slopes of the simultaneous longitudinal water profiles may be as much as 1 ft. per sea-mile.

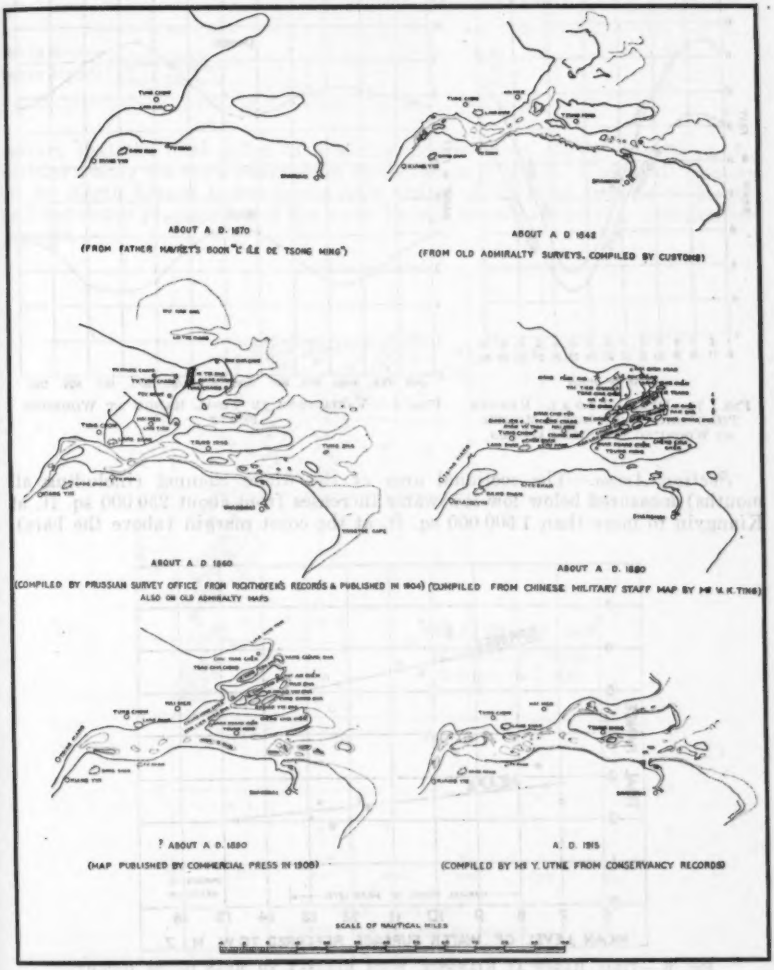

FIG. 6.—SERIES OF HISTORICAL PLANS OF THE ESTUARY.

Velocities.—Owing to the magnitude of the stream the observation of current velocities has not been very exhaustive, but it has been found that in most places the velocity near the bottom is about 60% of the mean velocity and that the mean velocity occurs at about 60% of the depth.

There are great irregularities in the velocity at different depths due to the lack of symmetry of the sections and the tidal-current reversals. The dis-

tribution across the stream shows nothing unusual in spite of the magnitude of the sections.

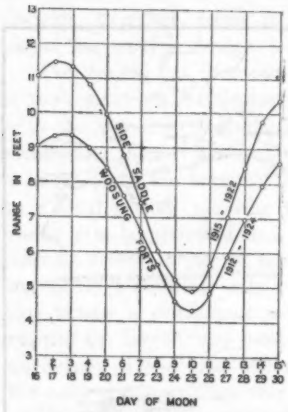

FIG. 7.—MEAN TIDAL RANGES THROUGHOUT THE LUNAR MONTH AT WOOSUNG AND SIDE SADDLES.

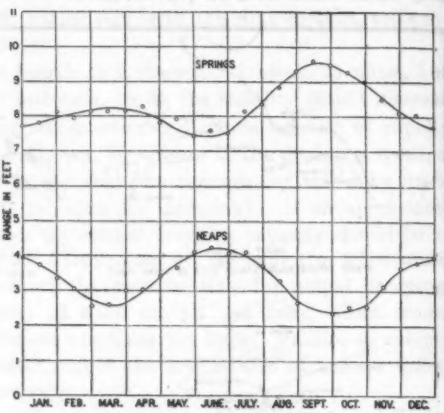

FIG. 8.—VARIATIONS IN TIDAL RANGE AT WOOSUNG FORTS.

Section Areas.—The sectional area of the whole channel (including all mouths) measured below low low water increases from about 250 000 sq. ft. at Kiangyin to more than 1 500 000 sq. ft. at the coast margin (above the bars).

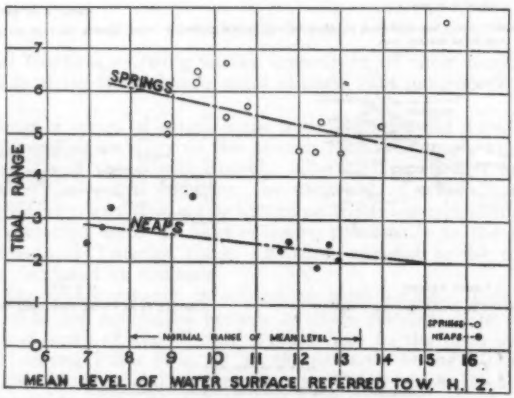

FIG. 9.—TIDAL RANGE AT KIANGYIN, WITH RESPECT TO MEAN GAUGE HEIGHT.

The sectional area below standard spring high water at the coast margin is about 3 000 000 sq. ft.

These areas are divided between the three entrance channels approximately, as shown in Table 2.

Discharges.—It has not been feasible to make a simultaneous observation of the discharge of all three channels owing to the number of large craft required, but from the various individual discharge measurements it appears

TABLE 2.

Channels.		Area below low low water, percentage.	Area below high water, spring tide, percentage.
North Branch		30	33
South Branch	North Channel	40	35
	South Channel	30	30

certain that the total influx of a spring tide is about 250 000 000 000 cu. ft. divided among the three channels in about the ratio, 2:3:3. The small amount in the North Branch is due to the later arrival of the tidal wave at its mouth and the slower propagation of the wave through its rather tortuous and shallow passage.

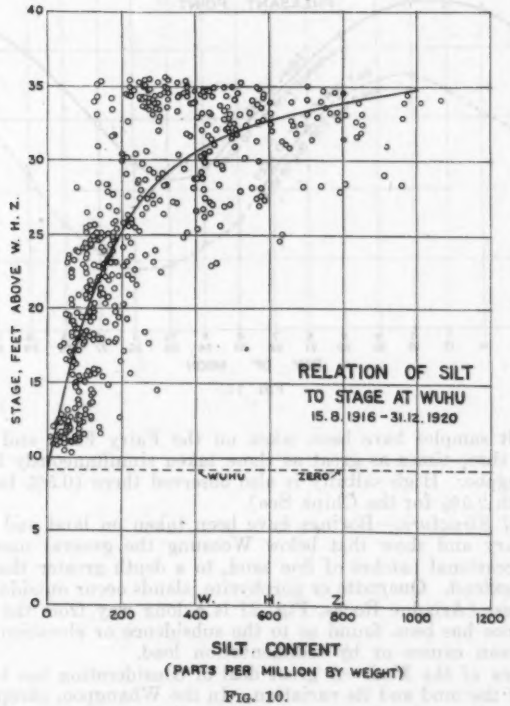

FIG. 10.

Tidal Magazine.—The capacity of the estuary is, of course, equal to the influx and diminishes for spring tides from about 200 000 000 000 cu. ft. at the coast line to 5 000 000 000, or less, at Kiangyin.

Run-Off.—From gaugings in the upper river it has been found that the fresh-water discharge of the river varies from about 250 000 cu. ft. per sec. to more than ten times that amount, averaging about 1 000 000. The maximum is in the late summer and the minimum, in January.

Silt.—Immense quantities of silt are brought down. Fig. 10 shows the silt charge at Wuhu (a point where tidal influence is very small, about 300 miles from the mouth), in relation to the stage of the river, which, of course, varies with the run-off.

In the estuary the tidal currents disturb and keep in suspension a great quantity of silt so that even in the winter, when the supply of fresh silt from the interior is small, the spring tides working in shallower channels develop more velocity and the water has a maximum of turbidity.

Fig. 11 shows the variation of silt with the tidal range at a point in the Whangpoo (Pheasant Point), quite close to the Yangtze.

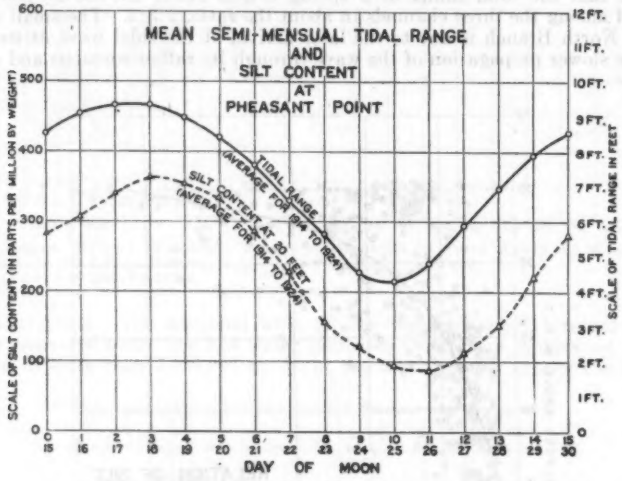

FIG. 11.

A few silt samples have been taken on the Fairy Flats and show values from one to three times as great as those taken simultaneously in the mouth of the Whangpoo. High salinity is also observed there (0.5% to 2.0% which compares with 2.5% for the China Sea).

Geological Structure.—Borings have been taken on land and under water in the Estuary and show that below Woosung the general mass is alluvial mud, with occasional patches of fine sand, to a depth greater than any works are likely to extend. Quartzite or porphyrite islands occur outside the Estuary, but the nearest (Ariadne Rocks, Fig. 2) is a long way from the bars.

No evidence has been found as to the subsidence or elevation of the delta by subterranean causes or by sedimentation load.

Consistency of the Mud.—A great deal of consideration has been given to the nature of the mud and its variation. In the Whangpoo, except in the case of recent accretion, it is a heavy material containing sufficient colloidal matter to make it very firm; but it cannot be regarded as a real clay. Silica predominates as the mineral constituent, and the mud is fairly easily disturbed by a stream of water; but when consolidated it presents considerable resistance to dredging by grabs or bucket dredges. The mud on the Fairy Flats closely resembles that in the Whangpoo.

Meteorological Data.—Ample meteorological data are available. The Fairy Flats is practically in the open sea, being sheltered only on the southwest.

The North Channel Bar is quite in the open. Northwest winds prevail in the winter and the bar is quite choppy during flood current. Southeast winds prevail in the summer and ebb currents are then the more disturbed. Very dangerous typhonic winds may occur in the late summer and autumn. Moderate or smooth weather occurs for about 80% of the time. About 100 hours of fog per year occur on the Fairy Flats and about 400 on the North Channel Bar.

DISCUSSION

A. W. Robinson,* M. Am. Soc. C. E. (by letter).—The writer, having visited Shanghai on several occasions and passed in and out over the bar, is able to visualize the magnitude of the problem and the character of the difficulties so graphically and frankly set forth by Mr. von Heidenstam.

The Board of Engineers' report of 1921 was largely concerned with the development of the Whangpoo as being Shanghai Harbor proper, and with the re-organization which it deemed necessary to enable the Conservancy Board to attack the great problem of deepening the approaches.

The political condition of the Central Government in China makes it hopeless to expect any aid from that source, as is usual in the United States and other countries, and it is, therefore, on the resources and revenues of the Port itself that the Board must depend for its execution and maintenance. Hence, it is of vital economic importance that the capital cost and maintenance charges of the work should not become so great a tax on the commerce of the Port as to burden or defeat the progress and growth of that commerce which it was designed to promote. This means that if dredging the channel is to be attempted, it must be done on a larger scale and at less unit cost than any other similar work in the world.

The Board of Engineers, in reporting on the subject of approaches, stated:

"The Committee is of opinion that deepening in the South Channel should be undertaken by dredging only, at least in the first instance, and that by this means there is a reasonable prospect of permanent improvement."

It also recommended,

"That the harbor authority should proceed to make and maintain a channel through the Fairy Flats developing it gradually foot by foot, to as great a depth and width as required and possible, and to provide as soon as feasible a channel of 600 ft. bottom width for the passage of ships drawing 33 ft. at ordinary neap high water."

This being the decision of this very competent Board of Engineers, the writer will not venture to criticize it or to suggest any alternative. However, there are some very important points to be borne in mind in considering such large-scale dredging plants, and in forecasting the probable results that may be obtained. These points are applicable not only to the work in question, but to all similar works throughout the world. That report was made in 1921, and nothing has yet been done, owing probably to political conditions; but it is presumably becoming more urgent every year.

General Method of Dredging.—In discussing the general method of dredging to be followed, Mr. von Heidenstam is of the opinion that a broad shallow cut has apparent advantages over a narrow deep one, and that the decision seems to rest exclusively with the drag-suction, self-propelled hopper type of dredge. With this the writer concurs, provided the material be suitable. It is better to make a shallow cut quickly across a bar in the direction of the current, in order that it may be self-scouring as far as possible.

* Cons. Engr., Upper Melbourne, Que., Canada.

Dredging on the Mississippi River has been done successfully for many years on this principle with a forward feed type of dredge having a wide suction mouth and anchorage lines up stream. The self-propelling drag-suction type of hopper dredge used for deepening ocean bars at New York and other ports also makes a continuous longitudinal cut, although of narrower width. The sand-pumping hopper dredges on the Mersey River, in England, however, work on a different principle, that of dropping the suction down in one place and making a conical depression as the sand runs in. The hopper is filled easily and quickly by this method, and it is supposed that the net result, after the several pockets thus made have evened out, is entirely satisfactory.

It is supposed that so many cubic yards removed from the channel must have the effect of deepening it. While this is undoubtedly true, it is nevertheless a fact that enormous quantities of sand have been removed from the Mersey without corresponding increase of effective depth. This has been attributed to the shifting nature of the sand, but the writer thinks that pocket dredging in this way offers an inducement for the sand to fill in again, whereas in rapid longitudinal cuts there is not the same tendency, and they are, in a measure, self-scouring. In all these localities the material dealt with is sand.

Character of Material.—This has an important bearing on the success or failure of hopper dredging. Sand, especially coarse sand, is ideal for the purpose as it precipitates quickly in the hoppers, filling them to the top while the water runs off in the overflow. However, sand varies much in coarseness and mud content, and more or less solid matter is washed overboard and lost. In very fine sand mixed with mud, as much as 50% or more may be lost in this way; and in alluvial silt or soft mud only some lumps not broken up will remain in the hopper. This loss may become so great that it would not be profitable to dredge it at all by this method.

At Brunsbuttel, on the Elbe, the silt is so fine and impossible to precipitate that it comes up in the consistency of soup, and when the hoppers are full the dredge goes to sea without any overflow at all. This means that much water is carried as dead load, but it is better than continuing to pump and losing it overboard.

Precipitation of Solid Matter.—Various methods have been tried to increase the precipitation of solid matter in the hoppers, such as baffle-plates and distributing troughs. In muddy material these are only partly successful, and it is obvious that the two essential conditions necessary to precipitate fine material in suspension are time and quiescence. In a dredge hopper these conditions are difficult to obtain. The incoming stream sets up currents and some agitation and time cannot be allowed for slow precipitation. The best that can be done, therefore, is to distribute the delivery into the hopper so that the least agitation will occur, and design the overflows so that the water is removed after the longest interval between the inflow and outflow both as to time and distance.

These conditions can be much better fulfilled in large hoppers than in small. Hence, a very large hopper dredge offers a better opportunity to

reduce the losses in overflow. If the mud will not precipitate in the hoppers sufficiently after pumping, then it will be necessary to consider some other means of filling them.

Type of Drag.—The ordinary form of drag on most of the hopper dredges on the Atlantic Coast (for use in sand) would be quite unsuitable on a packed mud bottom of a nature to require dredging by a bucket or grab. It would simply ride over it and leave a track mark. Such stuff as was picked up and churned by the pump into a fine solution of mud would not precipitate rapidly enough to make a load in any reasonable time.

For such clayey material it would be necessary to excavate it or break it up into lumps suitable to be handled by the pump in such a way that they would not be entirely churned up, but would remain in the hopper. This can be done by a toothed drag or plow having multiple teeth or plowshares, similar to the teeth of a farmer's plow or cultivator. The writer has designed a suction cutter for clay soils, in which multiple cutters or plows are arranged in front of the suction in such a way that all the pump has to do is to pick up the loose material. The cutters are constructed so that if any tooth strikes a snag it will drop back and ride over the obstruction.

Grab Type.—The grab or clamshell type of dredge is well suited for soft clay and the wear and tear and mechanical losses are less than with the pump type. A series of grabs could be arranged on the dredge which could load the hoppers with no loss by overflow and at any desired capacity rate. With electric distribution of power, oil-engine generated, the problem would be simplified and good economy obtained.

Economy of Large Units.—After the dredging the problem is one of transporting and dumping at sea. It is well-known that large vessels are more economical than small, both in propelling power and operating costs. The limiting factors in ocean vessels are draft of water and time of loading.

Table 3 was prepared by the writer for another paper.* It shows the comparative economy of cargo vessels of 2 500 to 25 000 tons dead weight capacity.

TABLE 3.—Comparative Economy of Cargo Vessels.

Dead weight capacity, in tons	2 500	5 000	10 000	15 000	20 000	25 000
Length of vessel, in feet	215	300	430	510	590	675
Beam of vessel, in feet	36	44	58	68	72	80
Draft of water, loaded, in feet	22	25	28¼	32	34	36
Speed, sea-miles per hour	11	11	11	11	11	11
Indicated horse-power	1 460	1 900	2 800	3 750	5 000	6 000
Coal per hour, in pounds	2 920	3 600	5 000	6 380	7 350	7 950
Coal per 100 ton-miles, in pounds	10.6	6.5	4.5	3.9	3.3	2.9

The dimensions given may be varied, of course, to suit different designs. For instance, if less draft is required the hull may have more length and beam; but the comparison is sufficient to illustrate the great saving in coal consumption in the larger sizes. Thus, a vessel of 2 500 tons consumes 10.6 lb. of coal per 100 ton-miles at 11 knots, while a vessel of 25 000 tons only consumes 2.9 lb. per 100 ton-miles at the same speed. In other words, a given

* "Economy in Ocean Transportation," by A. W. Robinson, *Proceedings*, Eng. Inst. of Canada, February, 1919.

amount of cargo is carried in the larger vessel with 27% of the fuel, and a saving of 73%, with intermediate sizes in proportion.

Size of Plant.—As to size and capacity of such a dredging plant, the writer does not offer any opinion at this time, except to state the possibilities. In the present case the limiting depth of water of 18 ft. could be exceeded at high tide and after the channel has been partly deepened. The vessel, therefore, might be built to carry, say, half load at 18 ft. and full load at 22 or 24 ft., which would permit the use of a vessel of 15 000 tons.

Stability of Channel.—The question of the stability of the completed channel would remain to be proved by time, whether the initial dredging were on a large or a small scale. A tenacious mud would apparently offer greater stability than sand or silt, and it is probable that maintenance costs would not be great. A comparison of annual surveys would indicate the present rate of change.

Report of Board of Engineers.—Perusal of the report of the Board of Engineers mentioned by the author discloses a difference of opinion as to procedure.

One member stated:

"I differ in being of opinion that the data available at present are not sufficient to justify me in making the dredging a definitive scheme of improvement. I recommend the carrying out of an experimental dredging before taking up the actual dredging work, in order to ascertain the feasibility of the latter, and under favorable circumstances, the amount of work needed for the maintenance of a deep water channel on the bar."

The other six members stated,

"The experimental dredging recommended will prove to be futile and it would be unsafe to base any conclusions on the success or otherwise, of the results which might be obtained thereby. They consider it dangerous to follow this proposal for the reason that failure to effect anything approaching permanent deepening would create the impression that a satisfactory channel over the Fairy Flats cannot be produced by dredging. They believe on the contrary, that there is no reasonable doubt about providing a greater depth by dredging, and that the depth so obtained can be maintained by the same means. The one condition they wish to lay down is that the dredging operations should be on a really extensive scale."

The writer thinks that there is some ground for both these contentions. On the one hand some experimentation would seem to be essential to determine the behavior of the material under actual conditions of dredging; the rate of precipitation in a hopper, if pumping is used; the losses in the overflow; the best type of excavating or plowing drag suction to be used; and, generally, whether sufficient material could be retained in the hopper to make the pumping method feasible. If it could not, a test of some other method should be made.

The material on the bar is described as "mud, fairly easily disturbed by a stream of water, but when consolidated presenting considerable resistance to dredging by grabs or bucket dredges." The author does not state whether any experiments have been made to determine the points mentioned.

On the other hand such an experimental plant would prove nothing as to the general results, in deepening the channel or its permanence, but it would

certainly yield useful data. The writer, therefore, is inclined to agree with the Minority Report as to the necessity of obtaining data on which to base a sound design suitable to the conditions before embarking on any large expenditure.

With these data obtained, there is no doubt that a seaworthy dredging plant, that could deal with the large quantities involved at a low cost, could be designed and built with every assurance of success.

R. L. VAUGHN,* ASSOC. M. AM. SOC. C. E. (by letter).—Under the conditions outlined in this clear and interesting paper, any one experienced in problems of this kind would probably agree that the most promising line of endeavor would be by dredging. Training works appear to be almost out of the question: First, on account of their magnitude and cost; second, because of construction difficulties; and, third, because of doubts concerning their efficacy and permanence. Quite often under such circumstances jetties merely push the problem a little farther seaward, so that, with improved equipment available, the tendency in the United States has been more and more to rely on dredges to create and maintain channels over ocean bars.

Hydraulic hopper dredges are best adapted to a sandy material. With silts, from 5 to 10 yd. are often pumped overboard for every yard retained in the hoppers. The net results on the channel will be excellent, however, provided there is a current to carry away the material thus placed in suspension. Using a certain special type of suction head, remarkably satisfactory results have been obtained in moderately stiff mud. Where the distance to the dumping grounds has been relatively short, costs as low as $0.03 per cu. yd. (hopper measurement) have been achieved, this figure being exclusive of capital and administrative items.

With anything approaching a moderately stiff clay, so far as the writer is aware, drag-suction equipment has been a miserable failure. It hardly seems possible, in a location consisting of shifting mud flats, that there should be any trouble from this source. However, the material is described as being very firm, although it cannot be classed as a real clay. It is further stated that considerable resistance is presented to grab or bucket dredges. A first-class bucket dredge is capable of excavating very hard material. Any substance which presents a marked resistance to such a tool might well prove impossible for a drag suction.

There seems at least a possibility that the pipe-line dredge has been dismissed without giving it sufficient consideration. The outstanding objection to the use of such equipment in the present instance is the exposure to rough water. It is quite true that a pipe-line dredge could not be operated in such a location satisfactorily if held in position by spuds. It might be entirely feasible, however, to dig on wires.

Another difficulty would be with the pipe line itself, which would need to be of special and rugged design. So far as the writer knows, no equipment which would exactly meet the present condition has ever been built, for the

* Cons. Engr., San Francisco, Calif.

reason that it would be quite expensive and that there was little or no demand for it; but there is no evident difficulty that could not be overcome.

The author states and the writer agrees that, other things being equal, such a tool would be preferable. One important possibility is that the material, if it is anywhere near as stiff and sticky as the description indicates, would pile up at the end of the pipe line to a considerable extent, thus forming a bank alongside the channel. Such a bank would be subject to attack by wave action, but, nevertheless, it would remain a very distinct shoal, and tend to perform the function of a training wall, for a considerable period.

One of the great advantages of a pipe-line dredge is that when it is working at all it is working continuously. Except when interrupted by serious storms, a properly designed plant under the given conditions should be able to work 22 actual pumping hours per day, 25 days per month (if shut down on Sundays). The author states that the weather is moderate and the water calm 80% of the time throughout the year. If there should be a 20% loss of time due to rough weather, this time could be partly used for plant repairs and maintenance, giving an actual pumping time of 440 hours per month, or about 5 200 hours per year.

A dredge of the seaworthy characteristics required could easily be designed to have a capacity of 4 000 cu. yd. per hour, giving a capacity of 20 000 000 cu. yd. per year. Such output may appear fantastic to one not familiar with the latest developments in plant of this kind, but it is not much beyond the range of the writer's own personal experience. For instance, he has seen 2 000 cu. yd. per hour pumped through 1 500 ft. of floating 22-in. pipe line, the material being a mixture of 60% stiff mud and 40% broken oyster shells, which is a combination not particularly easy to pump. This particular dredge had about 1 200 h. p. (900 kw.) on the pump shaft.

Even should the capacity be grossly over-estimated, due to lack of familiarity with the material and the local conditions, still two such units would certainly be able to greatly exceed this annual output, if they could be used at all. A unit such as is contemplated, together with all its attendant auxiliaries, could be built in the United States for a cost probably under $1 000 000. In this country the operating cost of such a unit, exclusive of capital and administrative charges, but including maintenance and renewals, should not exceed $50 000 per month. These expenditures, both capital and operating, are materially below those given by the author.

With such an increase in the dredging capacity possible, much of the force would be lost from the argument that the cut should be deepened gradually and uniformly throughout its entire length. Of course, there would be considerable silting in the cut, but probably not enough to affect seriously the problem. Certainly, also, a part of the material pumped would find its way back into the excavated channel, but not as great a part of the actual quantity pumped as would be the case with a hopper dredge.

Many practical details of such a dredging plant would require special study for solution. One of the most important elements would be the pipe line and the method of controlling it. The line should have as little excess

buoyancy as possible, as one with a large excess of flotation would pound itself to pieces in a seaway. The effect of the strong currents on the floating line should also be carefully considered; likewise the method of holding and moving it. Doubtless some of the floats would go aground on the spoil-bank, but with the large range of tide the line should float at high water and could be shifted ahead at that time. The moorings to hold a large dredge on a mud flat should be given special consideration; likewise the means for shifting—always remembering that these operations must be carried on regularly in rather rough water. For anything like satisfactory running time it would be necessary to re-fuel while dredging, which would probably preclude the use of anything but oil fuel. It is suggested that, for power, first consideration be given to a direct-connected Diesel drive for the pump, and Diesel-electric drive for the cutter, swinging engines, and auxiliaries.

The writer has been encouraged to advance his present suggestion by a consideration of the depths of water that seem to exist in the vicinity. A greater part of the area over which it is desired to dredge appears to be protected somewhat by a rather extensive stretch of comparatively shallow water to seaward. Under such conditions a very nasty choppy sea may exist, one quite dangerous to small craft, but different from the heavy destructive surf which may be encountered where there is deep water in the immediate vicinity of a shoal. If there should exist in the locality in question the heavy breaking seas to be found at times in such places as San Francisco Bar, Eureka Bar, or Humboldt Bar, on the Pacific Coast, for instance, then only a highly mobile plant, such as a self-propelled hopper dredge, could be considered. It might be that a combination of the two tools would give the best solution, a self-propelled dredge for use on the outer part of the bar where the exposure is greatest, and a pipe-line machine for use farther in where the expanse of shallow water had broken up the heavier seas.

In any case the advent of a typhoon is an entirely different matter. The approach of such a storm can be detected, and retirement of all plant should be made to the most sheltered spot within reach. This would necessitate the services of a very able sea-going tug which should be available at all times. No dredge that the writer can imagine could successfully ride out a typhoon while lying on the bar.

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Paper No. 1699

RE-DESIGNING CATAWBA STATION FOR SERVICE ON A LARGE TRANSMISSION SYSTEM*

By W. S. Lee,† M. Am. Soc. C. E.

With Discussion by Messrs. H. A. Hageman, H. V. Joslin, T. H. Hogg, and W. S. Lee.

Synopsis

This paper describes the re-design and re-construction of the first hydro-electric power station of the Duke Power System in the Carolinas, now (1927) comprising twelve water-power and six steam-power plants of an aggregate installed capacity of 749 925 kv-a., feeding a transmission system of approximately 3 000 miles of electric circuits.

This plant, located on the Catawba River near Rock Hill, S. C., had an original installation of eight rope-driven generators of 5 600 kw. total capacity. The head of water was 23 ft. Operation of the plant began in 1904, and power was transmitted at 11 000 volts to adjacent territory. After twenty years' operation of the plant, it was decided to raise the dam 47.6 ft., and the reconstructed plant is now operating under a maximum head of 70 ft. The new power-house installation consists of four vertical-shaft, water-wheel-driven generators of 60 000 kw. total capacity. The current is stepped up to 100 000 volts for transmission.

Since the beginning of the Twentieth Century the electric light and power industry has developed rapidly and a modern hydro-electric system now consists of a group of stations of various kinds, which may be described, as follows:

First.—Stations that use the fall and run of the river.

* Presented at the meeting of the Power Division, Asheville, N. C., April 21, 1927.
† Vice-Pres. and Chf. Engr., Duke Power Co., Charlotte, N. C.

Second.—Stations that use the fall and run of the river plus a small amount of storage, taking care of the night and Sunday flow and perhaps of minor weekly fluctuations of the flow of the river.

Third.—Stations that have a large storage capacity and are used as a valve or outlet to supply the deficiency in power of other stations during low-water seasons.

In addition to the hydro-electric stations, the present large power system usually comprises a number of steam-power stations which, preferably, are located at, or in proximity to, industrial centers so as to reduce the losses in transmitting the power to the points of delivery. They have various efficiency ratings, depending on the time they are intended to be operated in conjunction with the hydro-electric plants, and they may be divided into the following classes:

1.—Plants designed to operate intermittently and during low-water periods; termed "stand-by plants".

2.—Plants designed to operate twelve months of the year and capable of supplying a constant amount of power; termed "base plants".

Finally, by interconnecting the trunk lines of the transmission systems of large independent power companies operating in adjoining districts, energy can be transferred from one district to another in a so-called super-power zone, the principal object being to insure a reliability of service and to make possible a more effective utilization of the water power from streams having different characteristics of flow.

The Duke Power System, in the Piedmont Section of the Carolinas, is now (1927) operating twelve water-power plants and six steam-power plants which may be grouped as follows:

	Installed capacity, in kilovolt-amperes.
A.—Run of River Hydro-Electric Stations:	
Greatfalls	30 000
Dearborn	56 250
Total	86 250
B.—Hydro-Electric Stations with Storage Ponds:	
Lookout Shoals	22 800
Rocky Creek	30 000
Cedar Creek	56 250
99 Islands	22 500
Total	131 550
C.—Hydro-Electric Stations with Large Storage Reservoirs:	
Bridgewater	25 000
Rhodhiss	31 875
Mountain Island	75 000
Catawba	75 000
Fishing Creek	37 500
Wateree	70 000
Total	314 375

FIG. 1.—GENERAL VIEW OF THE OLD CATAWBA PLANT.

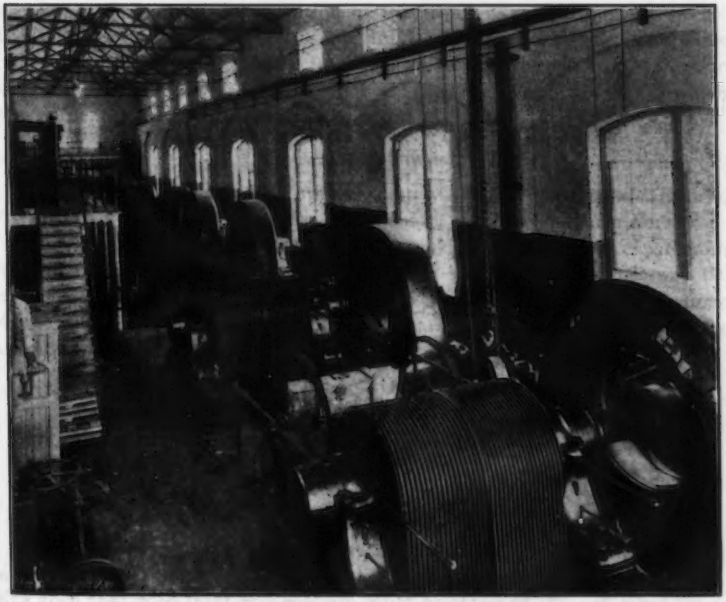

FIG. 2.—INTERIOR VIEW OF OLD CATAWBA STATION.

	Installed capacity, in kilovolt-amperes.
D.—Stand-by Steam Stations:	
Greenville	8 000
Greensboro	8 000
Total	16 000
E.—Base Steam Stations:	
Tiger	37 500
Mount Holly	45 500
Eno	31 250
Buck	87 500
Total	201 750

These power stations, of an aggregate installed capacity of 749 925 kv-a., feed into a transmission system consisting of approximately 3 000 miles of circuits, of which more than 50% are 100 000-volt, steel-tower lines. These power lines serve a territory approximately 350 miles long and 150 miles wide, and the trunk lines are interconnected with the transmission systems of three independent companies all of which operate in the Southeastern Power Zone comprising the States of North Carolina, South Carolina, Georgia, Alabama, and Tennessee.

In 1904, when the Catawba Power Company, now forming part of the Duke Power System, began operation of the 10 000-h. p. development on the Catawba River at India Hook Shoals, 6½ miles north of Rock Hill, S. C., three 11 000-volt pole lines, of 28 miles total length, carried the electric current to the Towns of Rock Hill and Fort Mill, S. C., and Charlotte, N. C., which latter point was 18 miles distant from the power plant.

Old Catawba Station

The first plant of the Duke Power System was called the Catawba Station (Fig. 1), and contained eight horizontal-shaft, rope-driven generators, four of 750 kw. each, and four of 900 kw. each, making a total rated capacity of 6 600 kw. (Fig. 2). The speed of the horizontal-shaft, triple turbines was 110 rev. per min., and that of the three-phase, 60-cycle, 11 500-volt generators, 300 rev. per min. The normal head of water at the station was 23 ft. and the guaranteed efficiencies were, as follows:

	Efficiency, percentage.
Turbines	80
Generators	94
Rope drive	94
Over-all efficiency	70.7

The highest flood on record at that time was 27.1 ft. above normal tail-water level, and the generator floor was 1 ft. higher, the station arrangement being such that good light and cool air could be obtained through windows on both sides of the power house.

The impounding structure of the plant consisted of a curved spillway, 585 ft. long, of standard ogee type, built of bulk concrete masonry, with an

average height of 34 ft. above the bed-rock foundation. An earth dam, 400 ft. long, with slopes riprapped on both sides, was rebuilt and raised after the July, 1916, flood, when the river discharge exceeded the 1901 flood by 100 per cent.

The drainage area of the Catawba River at the station is 3 085 sq. miles, the pond area was 1 800 acres, and the available storage capacity, 202 000 000 cu. ft. Before the large storage reservoirs were built on the Upper Catawba River, the stream carried large volumes of silt in suspension during flood stages, a considerable amount being deposited in the Catawba pond, so that after twenty years' operation of the plant barely the night low-water flow of the river could be stored when the plant was not operated 24 hours.

Following the July, 1916, flood, the Duke Power Company built three large storage reservoirs on the Catawba River above the station for the purpose of impounding the high waters of the Upper Catawba River for use during periods of low water on the Lower Catawba. This resulted in a large increase of uniform flow at the Catawba Station and the question arose whether additional machinery should be installed and the old plant overhauled; whether the dam should be raised so that the water would back to the tail-race of the Company's, Mountain Island Station, 25 miles farther up stream, thus creating another large storage reservoir; or whether an entirely new power house should be built and the old power plant representing a value of approximately $1 000 000, scrapped.

A topographic survey of the entire storage basin was then made, and it was found that for a development of 70-ft. head, which called for a raising of the old spillway 47.6 ft. above the top of the flash-boards, the superficial area of the full reservoir was 12 455 acres and the available storage capacity 10 300 000 000 cu. ft. considering a 30-ft. draw-down. In view of the fact that this storage was available not only for the new Catawba Station, but also for the existing stations of the Duke Power Company, on the Lower Catawba River, having a combined mean head of 250 ft., authority was given, in August, 1924, to raise the old dam and to build a new power station.

Designing Features of New Plant

As shown in Fig. 3, the new station is on the left side of the river, looking down stream, and the lower part of the south end of the substructure is built against the retaining wall for the old earth fill. The developed length of the new spillway is 802.93 ft. measured between centers of flood-gate piers. Referring to Fig. 4, the spillway contains eleven steel flood-gates, 45 ft. wide by 30 ft. high, and one smaller gate, 19 ft. wide and 25 ft. high. The curved section of the spillway, 168.52 ft. long, contains no flood-gates and the top is at Elevation 570.0, corresponding to the top of the flood-gates when in closed position. The space occupied by the masonry supporting gates, Nos. 9, 10, and 11, and the small gate, contains the substructure of the old power house.

The raised spillway follows the same alignment as the old spillway and the new masonry is built on top, against the down-stream face of the old masonry, as indicated on Fig. 5. Three 12 by 12-in. drains running along the rollway of the old structure are formed in the new masonry and are provided with 12 by 12-in. side outlets, at 55-ft. intervals, by which any seepage water from the

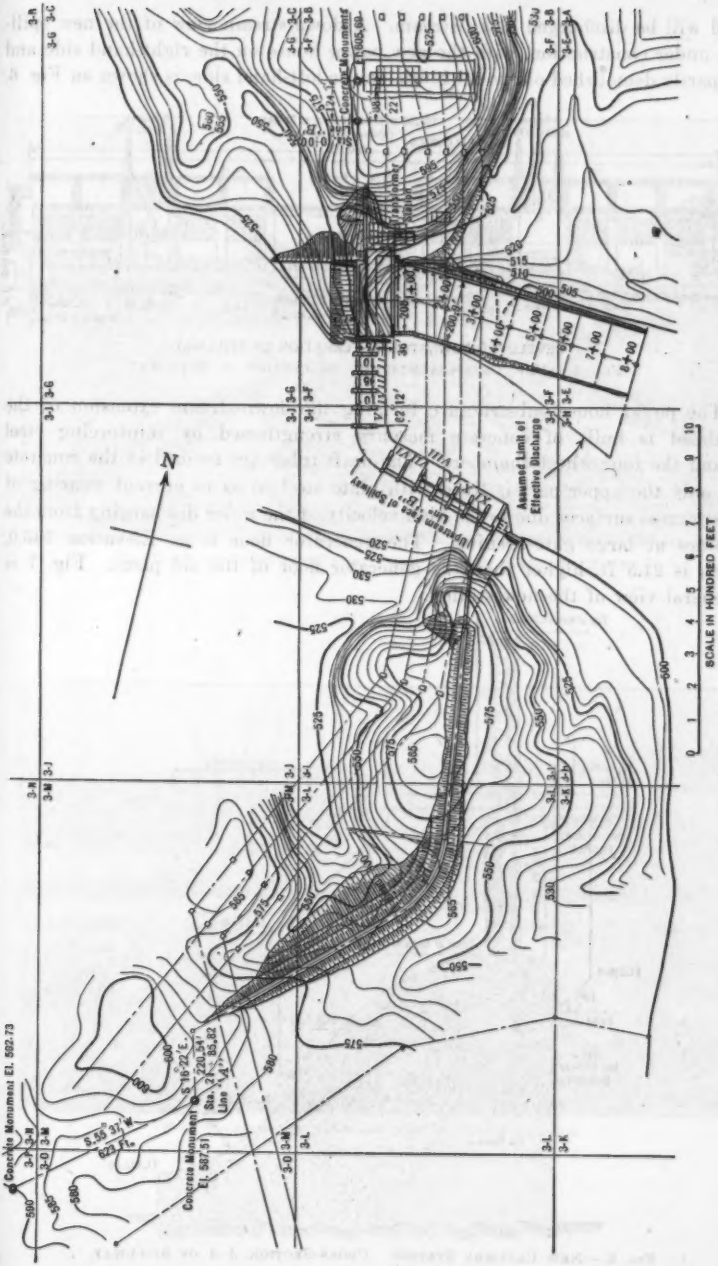

FIG. 3.—GENERAL PLAN OF NEW CATAWBA STATION.

pond will be discharged down stream. A down-stream view of the new spillway under construction, with the new power house on the right-hand side and the partly demolished old power house on the left-hand side, is shown on Fig. 6.

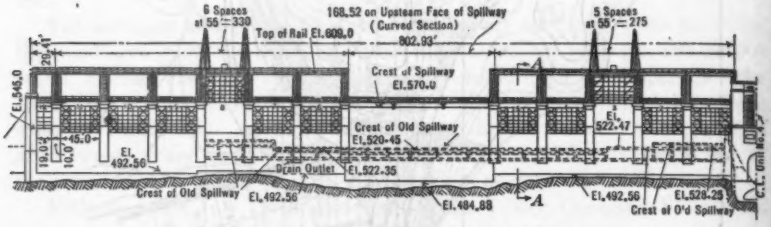

FIG. 4.—NEW CATAWBA STATION. ELEVATION OF SPILLWAY.

The power house substructure forming the down-stream extension of the bulkhead is built of concrete masonry strengthened by reinforcing steel around the four wheel chambers. The draft-tubes are formed in the concrete and only the upper part is lined with plate steel so as to prevent wearing of the concrete surfaces due to the high velocity of the water discharging from the turbines at large gate-opening. The generator floor is at Elevation 545.0, which is 21.5 ft. higher than the generator floor of the old plant. Fig. 7 is a general view of the new station.

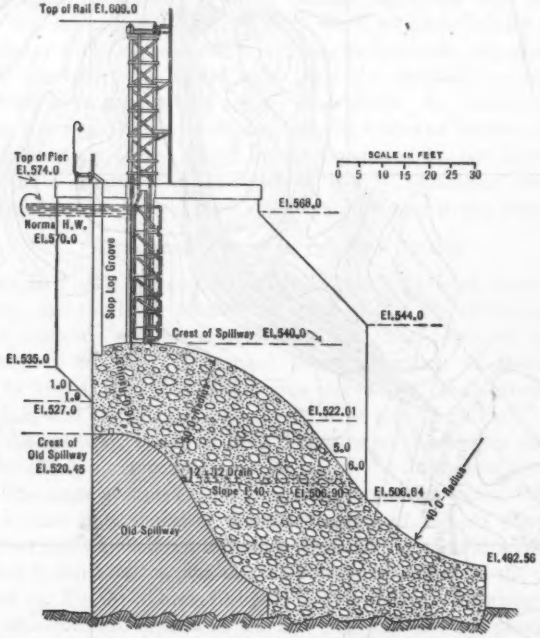

FIG. 5.—NEW CATAWBA STATION. CROSS-SECTION *A-A* OF SPILLWAY.

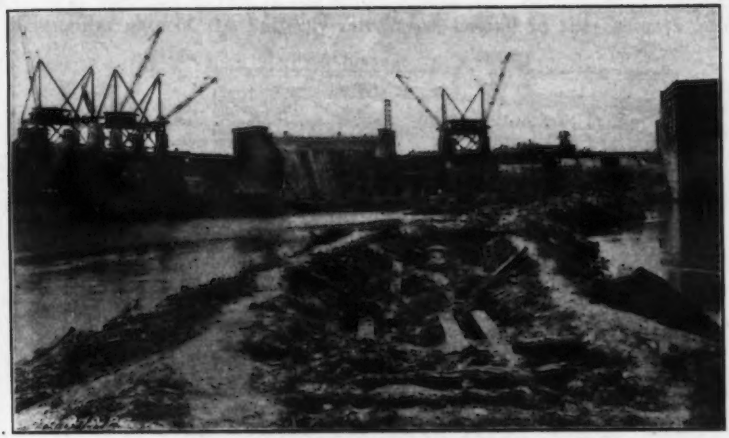

FIG. 6.—NEW SPILLWAY UNDER CONSTRUCTION, CATAWBA PLANT.

FIG. 7.—GENERAL VIEW OF NEW CATAWBA STATION.

The rack structure at the up-stream face of the power-house bulkhead is built of reinforced concrete and structural steel, the spacing of the rack bars being 4½ in., center to center. The intake to each turbine flume consists of two 18 by 25-ft. openings formed in the concrete masonry. Each opening contains a rectangular gate of the butterfly-valve type, having in the center a cast-

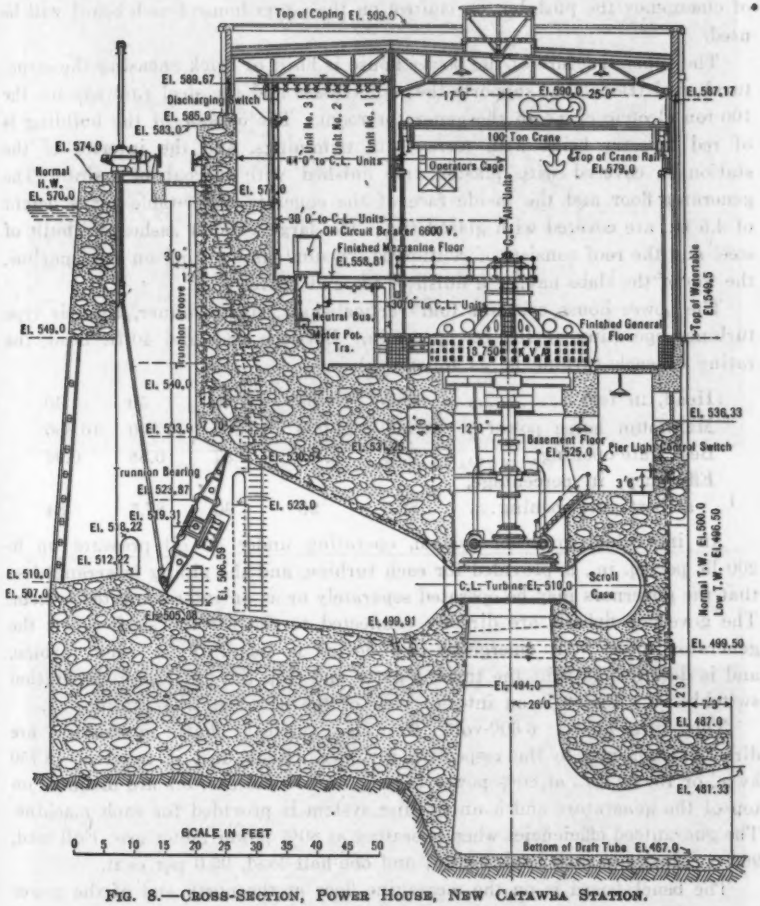

FIG. 8.—CROSS-SECTION, POWER HOUSE, NEW CATAWBA STATION.

steel girder, 3 ft. deep and 2 ft. 8 in. wide, with bronze-bushed trunnions, 28 in. in outside diameter and 20 in. long, at each end. The wings of cast iron are firmly attached to the girder by machine bolts and shrink rods. A built-up cast-iron wall-frame securely anchored to the masonry is provided with machine-finished surfaces at its lower half at which point the surfaces of the gate and wall-frame come in direct contact. The upper half of the wall-frame has machine-finished beveled faces conforming to similar surfaces of the gate. The slot between the upper half of the gate and the wall-frame is sealed by a

set of brass flaps attached to the up-stream side of the frame. Each gate is operated by an individual, completely enclosed, electrically operated hoist on top of the power-house bulkhead, as shown in Fig. 8. A gate-position indicator is provided on top of the hoist. Normally, the head-gate motor will be operated by means of a push-button control system on top of the bulkhead and in case of emergency the push-button control on the power-house bench-board will be used.

The superstructure of the power house is built of brick encasing the structural steel frame that supports the roof trusses and the steel runways for the 100-ton electric crane in the generator room. The outside of the building is of red tapestry brick with terra cotta trimmings, and the interior of the station is covered with plaster and finished with a coat of paint. The generator floor and the inside face of the concrete water-table for a height of 4.5 ft., are covered with glazed tile. The large window sashes are built of steel and the roof consists of reinforced gypsum slabs resting on steel purlins, the top of the slabs having a finish of tar and gravel.

The power house contains four vertical-shaft, single-runner, Francis type turbines operating at 100 rev. per min. Between 68.5 and 40-ft. head, the rating of each turbine is, as follows:

Head, in feet	68.5	65	60	50	40
Maximum horse power..	22 500	21 000	19 000	14 800	10 600
Best gate-opening.....	0.85	0.86	0.87	0.88	0.96
Efficiency, in percentage, at best gate-opening...	89.5	90	90	87.5	84

An individual governor system, operating under an oil pressure up to 200 lb. per sq. in., is provided for each turbine, and the piping is arranged so that the governors may be operated separately or as an interconnected system. The governor flyballs are directly connected to the turbine shaft below the generator rotor. An emergency-stop device is installed for each turbine, and is designed to shut the turbine gates and stop the unit from the station switchboard. Fig. 9 is an interior view of the new Catawba Station.

The three-phase, 6 600-volt, 60-cycle, vertical shaft generators are directly connected to the respective turbines. Each unit is rated at 18 750 kv-a., or 15 000 kw., at 80% power factor. Individual exciters are mounted on top of the generators and a unit oiling system is provided for each machine. The guaranteed efficiencies when operating at 80% power factor are: Full load, 96.3%; three-quarters load, 96.0%; and one-half load, 95.0 per cent.

The bench-board is on the mezzanine floor at the north end of the power house. Through four water-cooled, outdoor-type, transformers of the same capacity as the generators the current is stepped up to 100 000 volts and then fed into the transmission system of the Duke Power Company. A view of the outdoor transformer station nearing completion is shown in Fig. 7.

The tail-race is 800 ft. long, and if the four units are operated at seven-eighths gate-opening and under 65-ft. head, the discharge is 12 000 cu. ft. per sec. Under these conditions the mean velocity of the water in the smallest tail-race section is 4.0 ft. per sec.

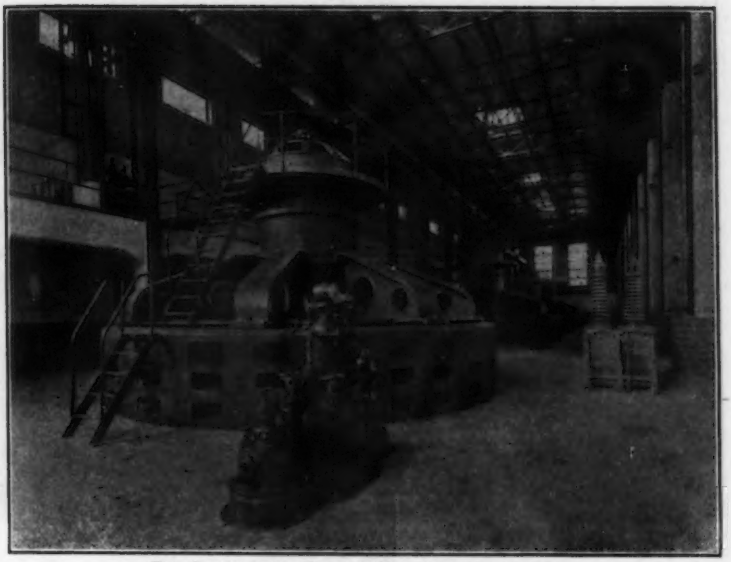

FIG. 9.—INTERIOR VIEW OF NEW CATAWBA STATION.

FIG. 10.—VIEW OF CONSTRUCTION TRESTLE ON TOP OF OLD SPILLWAY, CATAWBA STATION.

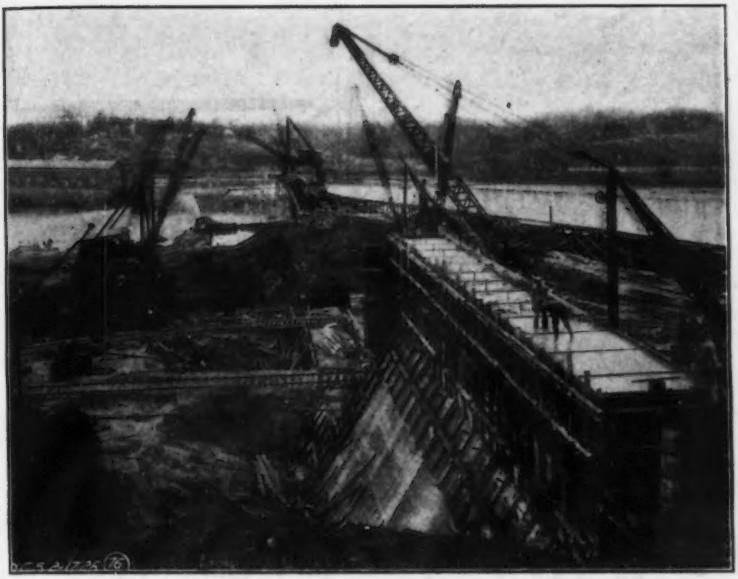

FIG. 11.—SITE OF NEW POWER HOUSE, SHOWING EXCAVATION WORK BELOW OLD EARTH FILL.

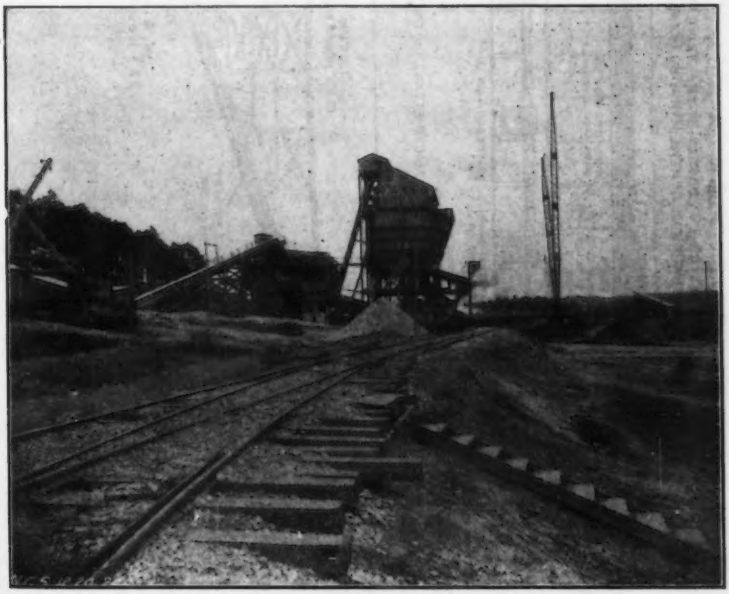

FIG. 12.—CRUSHER AND MIXER PLANT, CATAWBA STATION.

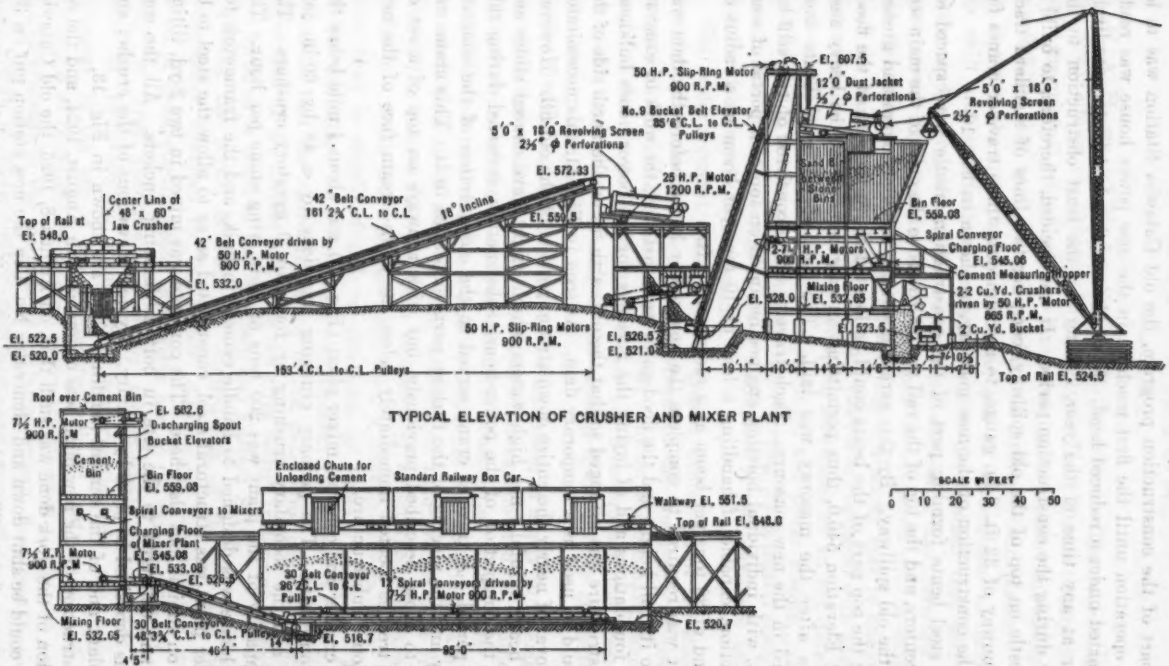

FIG. 13.—DETAILS OF CRUSHER, MIXER, AND CEMENT HANDLING PLANTS, NEW CATAWBA STATION.

Construction Features of New Plant

As part of the construction program, the old Catawba Station was to be kept in operation until the first machine in the new power house was ready to be started under a reduced head. It was known that high water was likely to occur at any time of the year, calling for the least obstruction to the spillway during the construction period. It was decided, therefore, to build a steel trestle on top of the old spillway, carrying two lines of standard track and a runway of 32 ft. 3-in. gauge, to accommodate three traveler cranes for use in the construction of the new masonry, as shown in Fig. 10.

The steel bents forming part of the construction trestle were spaced on 27½-ft. centers, and the top of the rail was 26 ft. above the crest of the main section of the old spillway. By this arrangement the track stringers and girders forming the top part of the bents could be located above the seat of the flood-gates at Elevation 540.0, thus permitting an easy removal of the heavy steel members after the masonry was laid. The legs of the steel bents were embedded in the new masonry. Each traveler crane consisted of two stiff-leg derricks, with individual hoisting engines mounted on top of a portal of steel construction, capable of handling a load of 10 tons per boom at a radius of 80 ft. and of 20 tons per boom at a radius of 40 ft.

As it was required to complete the new power house before the dam was raised to its full height and the flood-gates were installed, the work of excavating the foundations and of laying the masonry for the power house bulkhead and substructure was arranged so that the old earth fill on the north side of the river could be used as a temporary dam. As seen in Fig. 11, this necessitated the removal of nearly the entire down-stream portion of the old fill. However, neither heavy rainfalls nor high waters which might have caused slides and delayed the completion of the power-house substructure, occurred during this critical period. A traveler crane served in the construction of the concrete masonry and the placing of the turbine parts embedded in it. This crane was similar to those described previously, but with runways on top of a set of wooden trestles located immediately below the down-stream face of the new power-house substructure.

The crusher and concrete mixer plant (Fig. 12) was about ¼ mile below the power-house site. The primary crushing was done by a 48 by 60-in. jaw crusher and the secondary crushing by four No. 6 gyratory crushers. The rated capacity of the plant was 200 tons of 2½-in. ring stone per hour. The cement house was designed to handle cement in bulk, and the framework for the mixer plant was structural steel, constructed so as to allow the steel to be re-used on similar jobs elsewhere. The concrete was mixed in two 2-yd. tilting mixers and delivered to the site in bottom dumping buckets. All the concrete was deposited in place by gravity. Typical elevations of the crusher and mixer plant and of the cement-handling plant are shown in Fig. 13.

Construction of the new plant was commenced in August, 1924, and the co-ordination of the work done was such that, on August 8, 1925, the old Catawba Station could be shut down and Generator No. 1 in the new station put on the line two hours later. The old station was then dismantled and the new plant was entirely completed at the end of December, 1925.

DISCUSSION

H. A. HAGEMAN,* M. AM. SOC. C. E. (by letter).—The re-development of the power site on the Catawba River at India Hook Shoals twenty-three years after the original project was put into operation, is an excellent example of the possibilities that exist in modernizing old hydro-electric plants.

It shows the opportunities that are open to the engineer to make use of his skill and experience in creating a new development that will produce power and energy from the water more efficiently than formerly. Many water-power developments that have been in operation over a long period of years, have antiquated power-generating equipment that is uneconomical from the standpoint of output and maintenance as compared with present-day hydro-electric machinery. The hydraulic structures of an old development, such as the dam, head-works, water conduits, and power station, may be in good condition but the machinery for transforming the potential energy of the stream into electric current is often inadequate if the maximum amount of power and energy is to be obtained.

The re-development makes a much greater utilization of the stream flow than formerly and permits a large increase in the production of power and energy with considerable gain in efficiency. The increased capacity and energy output is due to the larger storage that has been created, the greater head used, and the higher efficiency of the modern hydro-electric generating and transmitting equipment.

The increase in head from 23 ft. to a maximum of nearly 70 ft. emphasizes the advantage resulting from a condition often encountered on Southern and Western rivers, of the absence of extensive construction in the river valleys. Fear of the oft-recurring floods and various other considerations have caused the inhabitants to do most of their building on high ground back from the river and to reserve the river-bank meadows for agricultural purposes. This results in a much lower flowage cost, when a material rise in water level is made, than is usually found on Northern or Eastern streams. Another distinct advantage to water-power development on many of the Southern streams is that the railroads, except in the mountains, generally follow the higher land of the divides and cross the river valleys instead of following the river bank, as is usual in the Northern and Eastern States.

During the period between the construction of the old and the new developments described by the author, considerable progress was made in the art of hydro-electric power generation and in the transmission of electric energy in large quantities. Probably the greatest advance was in the design of hydro-electric machinery; more particularly the water-wheel equipment, in the use of direct-connected vertical shaft units, and in the elimination of inefficient mechanical power transmission. It is also true that the improvement in the methods of analyzing stream flow, greater economy in the design of hydro-electric structures, and more efficient power trans-

*Chf. Hydr. Engr., Stone & Webster, Inc., Boston, Mass.

mission over long distances, have contributed to the progress realized. In view of this statement a comparison of the hydraulic features of the old and the new Catawba developments might be of interest.

The new station contains only one-half as many units as the old one, while the combined rated capacity is increased from 6 600 kw. to 60 000 kw. The new units are of the single, Francis runner, vertical-shaft type, each direct-connected to the generator, while the old units were of the horizontal-shaft type with triple Francis runners, each water-wheel unit delivering its power to the generator through a rope-drive. The over-all efficiency of the old units is stated as being 70.7%, while the combined efficiency of the new units is about 87%, an increase of 23 per cent. The generating equipment in the old station required about 0.73 cu. ft. per sec. to produce 1 kw., while in the new development, with the increase in efficiency and in head, the same result is obtained from about 0.20 cu. ft. per sec. under the maximum head, beside the large increase in output due to greater use of storage.

It is interesting to note that valves of the butterfly type, for water control, are used in the inlets to the new units. This type of valve has been used in water-power developments for a long time, but it is only within the last few years that it has been improved so that it could be used for large openings without excessive leakage. There is a great diversity in the type of control mechanism used and in the devices adopted to reduce leakage. It is understood that the Duke Power System has several installations of large butterfly valves, using the spring brass flaps attached to the up-stream side of cast-iron wall-frames to secure water-tightness. It would be of considerable interest to know how these have worked out in service, both as regards water-tightness and as to whether there has been any trouble from material lodging on these flaps. It would also be of interest to know the amount of power necessary to operate these gates under different conditions of flow, using the tension-rod type of control mechanism. The use of the direct-connected flyballs and of complete individual units in the governor system is typical of present practice.

It is evident that the flood conditions have been very severe on the Catawba River, since although the generator floor of the old plant was placed 28.1 ft. above the normal tail-water level, and 1 ft. higher than the maximum previously recorded flood level, the flood of 1916 submerged the station with a flow 100% greater than the previously recorded maximum flood. This seems a good reason for making the new generator floor level 21.5 ft. higher than the old floor level, or nearly 50 ft. above the normal tail-water level. This excessive flood condition is probably one explanation of the use of the very large flood-gates, 45 ft. long by 30 ft. high, although there may be other reasons for selecting these large gates in place of a greater number of smaller ones.

From the data given in the paper, it is noted that the usable storage has been increased from about 5 000 acre-ft. to about 236 000 acre-ft. This new storage changes the power and energy aspect of the possibilities at this site. Formerly, there was only sufficient pondage available to enable the flow to meet the daily load factor of the station, but now there is a large

regulated flow available, which can be used efficiently during periods of drought.

The lack of natural lakes and storage basins on the rivers in the southeastern part of the United States makes all artificial storage especially valuable, and the relative increase in the primary flow of the stream is often much greater than with an equal amount of additional storage developed on rivers with equal drainage areas in the Northern States.

The development and use of the large storage basins must have a considerable effect on the peak floods of the river and must be a valuable asset to all property owners along the lower valley.

It has been stated that large quantities of silt have been carried by the stream during high-water periods before the development of storage reservoirs. Any definite data regarding the lessened quantity of silt carried with the present storage conditions, or the rate of deposition of silt in the various storage basins, would be valuable. It is understood that this river is used for water supply in several instances. There must be considerable benefit from having clearer water.

The growth of the system load has apparently been very rapid, requiring frequent increases in power facilities, and this particular re-development may be taken as a notable achievement of present-day practice.

The tremendous increase in the size of single units that manufacturers are able to build, has given a great impetus to re-development of old plants. The use of the present modern reaction and propeller type runners, efficient vertical thrust bearings, improved draft-tubes, and large control gates, have all been factors contributing to the construction of more efficient and larger single units. Some of the points mentioned are illustrated by the following hydro-electric developments.

At the Garvin's Falls Plant of the Manchester Traction, Light, and Power Company, of Manchester, N. H., on the Merrimack River, the old plant consisted of six horizontal-shaft, triple-turbine, direct-connected units, under 30-ft. head. These were installed about 1900, and two units had new horizontal-shaft runners installed about 1920. In 1924, the four old, 800-kw., horizontal units were removed and two new vertical-shaft, propeller type turbines with direct-connected generators were installed. These new units are 3 000 and 4 000 kv-a., respectively. They increased the station capacity 50% with a corresponding increase of only 15% in the quantity of water required. The turbines are equipped with plate-steel Hydraucone draft-tubes, direct-connected governor flyballs, and adjustable lignum-vitæ guide-bearings.

In connection with this re-development, the plant at Kelley's Falls, on the Piscataquog River, was built over from a two-unit, horizontal-shaft, rope-drive plant, very similar to the Catawba Plant, to a single-unit, automatic, vertical-shaft, propeller type, direct-connected plant.

Another instance of the utilization of the power available under a very low head by means of propeller type runners is the Green Island, N. Y., development of Henry Ford and Son, Incorporated. This plant has a normal head of 13 ft., which is considerably reduced during high water, and contains

four 2 000-h.p. wheels, having 4-blade runners 156 in. in diameter, in a siphon flume setting.

A recent development containing modern, vertical shaft, Francis turbines, each 25 000 h.p., under 112-ft. head, and direct-connected to alternating-current generators, is the Bartlett's Ferry development of the Columbus Electric and Power Company. These units have elbow type draft-tubes.

An interesting example of the application of a butterfly valve for use as a head-gate to a water-wheel unit will be found in the Conowingo development now (1927) under consideration for The Susquehanna Power Company. Each water-wheel unit, which will have a rated capacity of 54 000 h.p. at full gate and 89-ft. head, will be equipped with a vertical shaft, cast-steel butterfly valve, 27 ft. in diameter, provided with specially designed rubber sealing devices. These valves will be the largest ever constructed and will be located directly at the entrance to the scroll-cases. Before selecting this type of valve a careful investigation was made of all types of head-gates with a view to securing the most satisfactory installation from the standpoint of cost and operation. The valves, as installed, will insure the least leakage of any type of gate under the particular conditions. They will be operated from the power-house floor and can be installed or dismantled by the main station cranes. This type of valve has permitted the most economical power-house design.

H. V. Joslin,* M. Am. Soc. C. E.—There is no question but that terracing the arable land would be of advantage to the power companies in that it would diminish silting in their reservoirs; but the speaker does not think missionary work of this nature among the farmers by agents of the power companies would pay for itself for some time to come.

Mr. Lee mentions the flood of 1916, when the river discharge was twice as much as had been heretofore recorded. This flood was considerably greater per square mile of drainage area on the Catawba than it was on the Pee Dee, where the Blewetts Falls Station of the Carolina Power and Light Company is located. The drainage area at the latter point is about twice that at Catawba, and, after data were received from the Southern Power Company of the discharge at Catawba, preparations were immediately made by the Carolina Power and Light Company to strengthen the Blewetts Falls dam and power house in anticipation of a similar flood.

It was discovered that in the event of such a flood the power house itself would probably float, and it was decided to anchor it to the rock below by drilling several holes about 35 ft. long through the concrete, and grouting reinforcing rods into place. Tripod drills of the largest type were used, and steel was obtained from Norway. Several of the holes had to be drilled through the generator-room floor through which a number of conduits had been placed, presumably according to blueprints that were available. Special precautions were taken to avoid hitting these conduits, but without success, and had it not been for the fact that the plant was idle most of the time, due to lack

* Engr., Carolina Power & Light Co., and Affiliated Cos., Raleigh, N. C. Mr. Joslin died November 3, 1928.

of water, the job would doubtless have cost considerably more. The water shortage was caused by the filling of the reservoir of the Tallassee Power Company farther up the river.

There has not been any flood in the river since, that has approached that of 1916 in intensity, and, inasmuch as since that time three large storage reservoirs have been built above Blewetts Falls, there will probably be no opportunity to test the efficacy of this anchorage. Mr. Lee's paper is very valuable, particularly at this time when, due to the great increase of efficiency in modern turbines and generators, many of the older hydro-electric plants are becoming obsolescent and sooner or later will require drastic remodeling. Probably not all of them will be rebuilt as completely as was the Catawba Station.

T. H. Hogg,* M. Am. Soc. C. E.—Hydro-electric problems in Ontario, Canada, are somewhat different from those in the region to which this paper applies. High flood flows are not experienced as frequently, and more extensive lake systems tend to equalize the flow. To take a specific example: on one of these river systems the drainage area is between 9 000 and 10 000 sq. miles, and lake systems thereon are so extensive as to permit almost complete regulation over a 5-year low-water period. The total run-off per square mile is less than in the South, amounting to 0.6 to 1.3 cu. ft. per sec. in this system.

There are some features of the paper that are of special interest in connection with the present situation in Ontario, as similar problems of load growth and of re-design and rehabilitation of plants are to be met. This is particularly true of the Niagara System, which covers the southwestern part of the Province and is fed by plants situated at Niagara Falls. The Hydro-Electric Power Commission controls three of the four large developments on the Canadian side of the river there; that is, the Toronto Power Plant (having a capacity of 150 000 h.p.), the Ontario Power Plant (180 000 h.p.), and the Queenston Plant (550 000 h.p.). The first unit of the Queenston Plant came into service in the spring of 1922. Nine units are now installed, and in December, 1926, the plant carried a load of 540 000 h.p.; that is, an increase in load of this same amount had taken place in the five years since the plant went into service. It will be seen from this that the problem of providing for increasing loads is strikingly similar to those outlined by the author.

Problems of design are somewhat different from those of the Catawba Station. The Toronto Power Plant was one of the first built at Niagara Falls, and, while the gross head is about 180 ft., the operating head, on account of large draft-tube and tail-race tunnel losses, is only approximately 135 ft. The efficiency of the development, based on the gross head, is less than 50 per cent. At Queenston the over-all efficiency is 91%, based on a 305-ft. head. In other words, Queenston delivers 30 h.p. per sec-ft. of water used, and the Toronto Power Plant delivers less than 10 h.p. International agreements fix the quantity of water that may be used to develop power, which means that to obtain additional power it will be necessary to curtail the output of the low-head and low-efficiency plants and use the water thus released more

* Chf. Hydr. Engr., Hydro-Electric Power Comm. of Ontario, Toronto, Ont., Canada.

effectively. This, within a short time, may compel the shutdown of the Toronto Power Plant, unless re-design and rehabilitation can materially modify its water economy.

Changes in the Toronto Power Plant made recently have reduced operating costs. The thrust-bearings in use formerly were of the oil-pressure type, operating at a pressure of 350 lb. per sq. in. These have been replaced by Kingsbury bearings, resulting in an annual saving of $15 000.

Certain operating records from Queenston may be of interest. This plant now (1927) operates on a load factor close to 90% for the whole year. The output amounts to 8 500 000 kw-hr. per day, and the writer believes that in the next two or three years this amount may be increased to 9 000 000 kw-hr.

One of the important operating problems on the Niagara River plants is that of ice control and disposal. The Niagara River is, of course, an open river at all times, but it carries enormous quantities of ice, which appears first about the middle of December and is sometimes present as late as the first of June. Ice flow during April and May, however, is not a menace on account of the high air temperature.

Protective equipment at the plant intakes, such as curtain-walls, overflow weirs, and ice runs, is used, and removes as much of the river ice as possible; but at the Queenston Plant, between the intake and the power house, there is 4½ miles of river section of canal, and 9 miles of excavated section. In addition, there flows into the river section the Welland River, about 45 miles long, which drains the country to the west of the Niagara River. Thus, even if all the Niagara River ice is excluded from the intake, it becomes necessary to cope with the ice formed in the Welland River and the long canal. As a matter of fact, the protective measures at the Queenston intake do exclude practically all the river ice, but at the Toronto Plant a considerable quantity of frazil and slush ice passes directly into the penstocks. The greater part of the ice reaching the head-works at Queenston, that is, the ice formed in the canal, is also of the same type, together with a comparatively small quantity of cake ice. The quantities of ice are so great that if it were not cared for in some way the plants would be shut down in an hour or two. In the first year of operation at Queenston, when the intake works were not complete, large quantities of ice gathered and at times curtailed the plant output appreciably, but since 1925 interruptions to the extent of a single kilowatt have not occurred.

If an attempt were made to by-pass the ice by means of the ice chute, which is provided at the end of the forebay, there would be a considerable abstraction of water that has been drawn through the canal, and a power loss of perhaps 30 000 or 40 000 h.p. The alternative is to allow the ice to pass through the turbines, and this has been done, obviating any curtailment of load. The advantages are so great that in the writer's opinion there is no question as to its being the proper solution.

The same method of disposing of the frazil and slush entering the Toronto Plant is followed there. The racks are removed, as otherwise they would quickly become clogged with ice and shut off the flow to the penstooks.

Too much emphasis can be placed on the necessity of keeping ice out of the plants. With the large machines now available, wheel passages are not readily blocked by the ice flowing in the rivers, and little danger can be involved in the method of operation outlined.

W. S. LEE,* M. AM. SOC. C. E. (by letter).—In the case of the Catawba Station the dam was raised and the power house was rebuilt, thereby obtaining a higher head, a greater utilization of the stream flow, and an increased efficiency due to improved design of the plant and equipment. While it is true that not all the older hydro-electric plants will be rebuilt as completely as the Catawba Station, the time will come when all the water powers of economic value are developed and the rehabilitation of the older plants must be considered if only for the reason of increasing the over-all efficiency and reducing the operating costs.

Mr. Hageman inquires as to the use of large flood-gates for the new Catawba development. The length and alignment of the old power house and spillway determined the effective length of the new spillway, and in order to take care of the extreme flood discharge it was necessary to consider flood-gates of such large dimensions. The wide openings have also the advantage of offering no restriction to large débris and trees which may be carried by the river at periods of high water.

As to the sealing devices for the head-gates, there is some slight leakage, but the drain provided at the lowest point of each turbine flume is more than ample to take care of this flow.

In the storage reservoirs of the Duke Power System no systematic quantitative measurement of silt deposited during high-water periods was made, and, therefore, no definite data are available.

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Paper No. 1700

UNUSUAL ENGINEERING FEATURES OF AN IMMENSE THEATRE BUILDING*

By R. McC. Beanfield,† Assoc. M. Am. Soc. C. E.

With Discussion by Messrs. Theodore Belzner, Fred A. Noetzli, A. Floris, Laurence J. Waller, and R. McC. Beanfield.

Synopsis

Some of the outstanding features of Al Malaikah Temple, at Los Angeles, Calif., are:

Combination pin and riveted balcony truss, 168 ft. span, weighing 250 tons;
Steel balcony cantilevers with a 45-ft. over-hang;
Stage with double asbestos fireproof curtain, probably the largest constructed to date (1927);
Reinforced concrete stage roof arches supporting steel gridiron which functions as a tie for the roof arches;
Seating capacity of 6 550 in orchestra and balcony;
Reinforced concrete proscenium arch, clear span of 100 ft.;
Auditorium steel roof trusses, one end on rollers, each weighing 60 tons; span, 192 ft.;
Banquet hall reinforced concrete roof arches, span, 90 ft.;
Ventilating system supplying 750 000 cu. ft. of air per min.; and
Stage switchboard and main chandelier, probably the largest yet constructed.

General Description

The Al Malaikah Temple in Los Angeles, commonly known locally as the Shrine Civic Auditorium, is the new home of the ancient Arabic order, Nobles of the Mystic Shrine. It is located at Jefferson, Royal, and Thirty-second Streets, the site of the old Shrine Auditorium destroyed by fire on January 12, 1920.

* Published in December, 1927, *Proceedings*.

† Cons. Engr., Los Angeles, Calif.

FIG. 1.—AIR VIEW, AL MALAIKAH TEMPLE, AUDITORIUM IN FRONT, BANQUET HALL AT LEFT.

FIG. 2.—VIEW OF INTERIOR OF THEATRE, SHOWING ABSENCE OF COLUMNS. CHANDELIER IS 22 FEET IN DIAMETER, GIVING IDEA OF SIZE.

The Temple occupies nearly 3 acres (Fig. 1), and may be divided into three units, as follows: The Auditorium, or theatre (Fig. 2), with facilities for housing the various Shrine organizations, such as Patrol, Band, Chanters, etc.; the Banquet Hall (Fig. 4), with a large basement for public exhibition purposes; and a kitchen wing, with facilities for serving large banquets. (See floor plan, Fig. 3.)

The construction throughout is reinforced concrete, except the roof trusses over the Auditorium, the trusses supporting the balcony, and the gridiron over the stage, which are steel.

The Auditorium

The Auditorium Section of the Temple is 200 ft. wide and 294 ft. deep, including the stage. It consists of a reinforced concrete frame with exterior concrete curtain-walls. The roof is constructed with a reinforced concrete joist system, which is supported on structural steel purlin beams connected to the steel roof trusses.

The Auditorium on the south side contains ample dressing-room facilities, including suites for stars. Large dressing rooms for male and female choruses are located in the basement under the stage. The musicians have a rest room and locker facilities. All the dressing rooms are efficiently ventilated and have shower baths.

The Potentate, Divan, Veterans, and various auxiliary organizations of the Shrine, such as the Patrol, Chanters, band, etc., are provided with complete facilities on the second floor on the south side of the Auditorium, which include a kitchen and large dining room.

The Pavilion

The north wall of the Auditorium is equipped with steel rolling fire doors, so that it can be shut off entirely from the Banquet Hall when desired. The large Banquet Hall, or Pavilion, contains 101 000 sq. ft. of floor space, including a large basement used for exhibition purposes; also, ground and second floors with a surrounding cantilevered balcony. There are two adjoining kitchens and serving rooms of capacity large enough to accommodate 10 000 people. The reinforced concrete roof of the Banquet Hall (Fig. 4) is supported on a series of beautiful reinforced concrete Saracenic arches, having a clear span of 90 ft.

The entire ground floor of the Pavilion is finished with white maple flooring, laid on a patented floor filler, called "Nailcrete". The Pavilion is used for art exhibits, civic expositions, and large balls. It has a clear ceiling height of 17 ft. in the basement, where machinery or automobile displays can be installed. Automobile ramps are provided from the Thirty-second Street front to this level. The basement is ventilated by a forced circulating system, the foul air being exhausted at the floor level in order to accommodate automobile exhibits.

The Stage

From a theatrical viewpoint, the Auditorium is one of the largest constructed to date (Fig. 2). Its stage, 72 ft. deep and 192 ft. wide, one of the

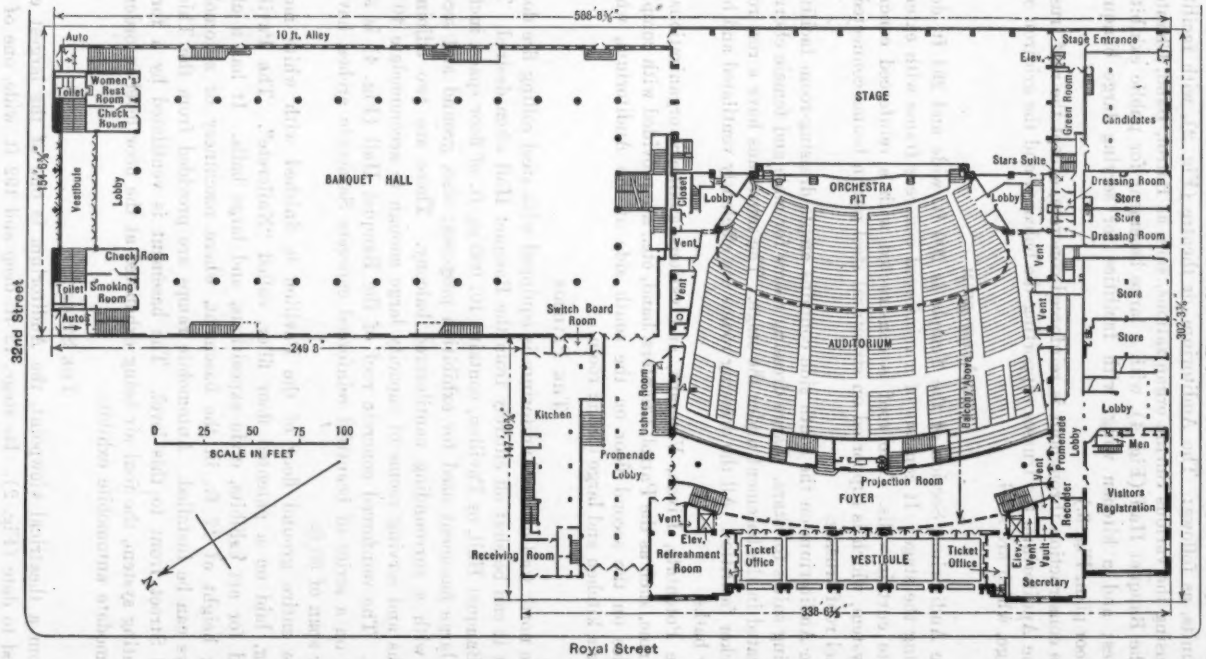


FIG. 3.—FIRST FLOOR PLAN, AL MALAIKAH TEMPLE.

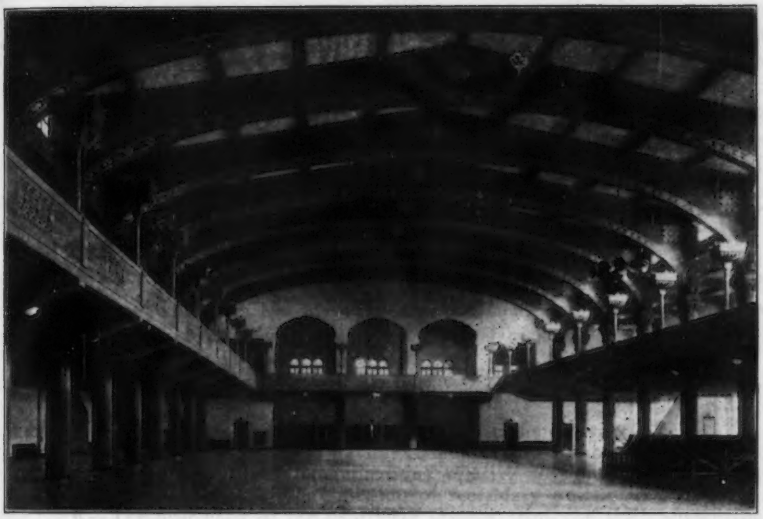


FIG. 4.—CONTINUOUS ROOF GIRDERS IN BANQUET HALL, SPAN 90 FEET. ANCHOR ARMS PROJECTED ABOVE LOWER ROOF LEVEL. DECORATIONS APPLIED DIRECTLY ON CONCRETE.

FIG. 5.—PROSCENIUM ARCH, 100-FOOT CLEAR SPAN. ASBESTOS CURTAIN AND LOUD SPEAKER HORNS ARE CONCEALED WITHIN THE JEWELED CROWN OF ARCH.

largest in the world, was designed to give ample room for the elaborate rituals of Shrinedom, as well as for grand opera and great theatrical spectacles. The orchestra pit can accommodate 200 musicians. The proscenium opening (Fig. 5) has a clear span of 100 ft. and a height of 37 ft. at the crown. The main floor seats 3 200, and the balcony, 3 350. With the exception of 22 seats behind balcony columns, the entire audience has an unobstructed view of the stage. The stage has a seating capacity of 1 700. It will be noted that the balcony seating exceeds that of the main floor.

Balcony Trusses

Preliminary designs and estimates were made for three different systems of truss framing under the balcony (Fig. 6). Scheme *C* was recommended as being the most practical, economical, and rational because:

1.—It contained less tonnage.
2.—Only two steel columns were required as compared to four in other designs.
3.—It contained fewer auxiliary trusses than the others.
4.—It was easier to erect, having fewer units and simpler connections.
5.—There were fewer complications relative to deformation, due to simplicity and the omission of secondary units.
6.—The over-hang of the cantilever trusses, from the main supporting truss, was practically the same, tending to provide uniform deflections in the cantilever trusses, which condition was very desirable, considering the reinforced concrete balcony deck construction.
7.—The entire design was rational, and involved no extraordinary features.

The main balcony risers and treads are reinforced concrete, supported by eight steel cantilever trusses (Fig. 7), having a minimum over-hang of 45 ft., which, in turn, are supported by a main balcony steel truss (Figs. 7 and 8). This truss is, doubtless, the largest steel balcony truss yet constructed for the purpose. It weighs nearly 250 tons, and has a clear span of 168 ft. It is of interest that the balcony truss columns were located to displace a single seat on each side of the Auditorium nearest the aisles adjacent to the walls (shown at *a*, Fig. 3), instead of outside the Auditorium walls, which arrangement would have increased the span 20 ft. The obstructed view for a few seats is relatively unimportant as compared with the increased difficulties in design and greater cost if 20 ft. had been added to the span of the main balcony truss.

As it is necessary to have sufficient clearance through the center part of this main truss for the balcony exit passageways, pin joints and eye-bar diagonals were used instead of the usual built-up members and riveted joints with larger gusset-plates. Riveted joints were used where practicable to obtain general stiffness. Where pins were necessary, nickel steel was chosen particularly to reduce the size of the pin-holes in the lower chord.

Steel Stresses

To avoid excessive moments and restraint due to the large angle of rotation in the truss, a pin-column joint was used. An analysis of the column joint based on a coefficient of friction of 0.25, indicated that the column must

resist moment due to pin friction before the pin could rotate. Few data are available relative to the friction of pins in steel trusses. This coefficient (taken from Professor Föppl) is, at best, only an approximation for this condition.

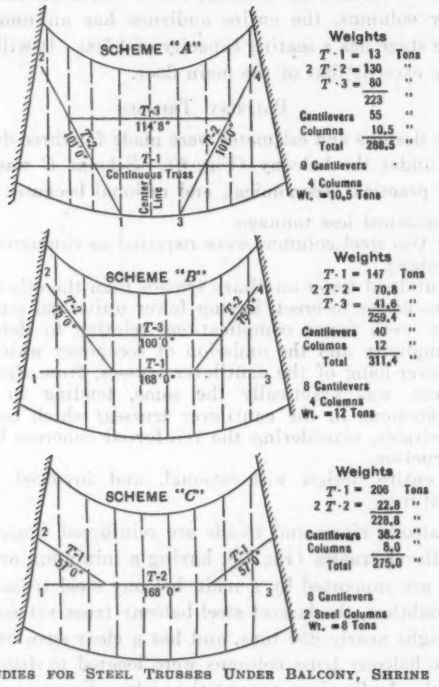


FIG. 6.—STUDIES FOR STEEL TRUSSES UNDER BALCONY, SHRINE AUDITORIUM.

Except for the stresses, the design of the balcony truss was based on the report of the Society's Special Committee on Specifications for Bridge Design and Construction.* The stresses recommended by the American Institute of Steel Construction were adopted. The stresses used for nickel-steel pins were 36 000 lb. per sq. in. in the extreme fiber for bending, 30 000 lb. per sq. in. for bearing, and 20 000 lb. per sq. in. in shear.

SECONDARY STRESSES

Due to the unprecedented size and unusual design of the main balcony truss, *T*-2, with its combination pin-and-riveted connections, a careful analysis of the secondary stresses was made. The computed restraining moments at the pin joints were rather uncertain as the coefficient of friction of the nickel-steel pins was an unknown factor and had to be assumed.

* Final Report of the Special Committee on Specifications for Design and Construction of Steel Railway Bridge Superstructure, *Transactions*, Am. Soc. C. E., Vol. LXXXVI (1923), p. 471.

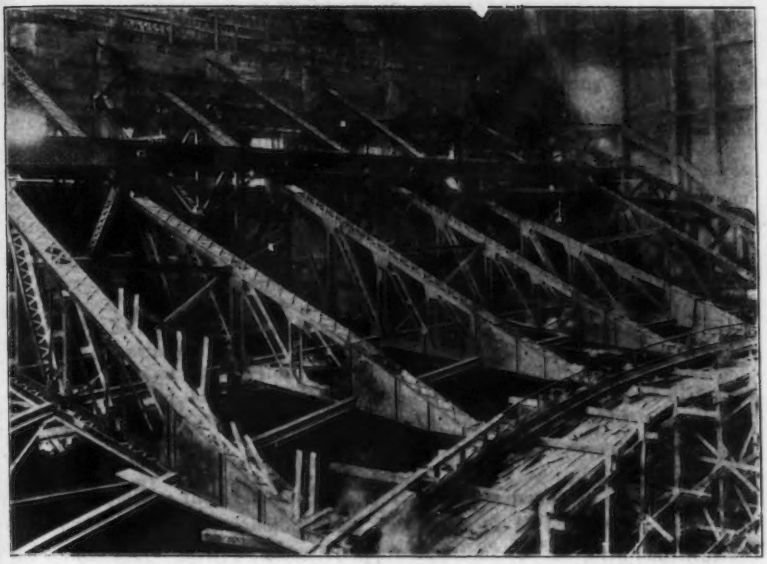


FIG. 7.—BALCONY TRUSSES CANTILEVERED FROM MAIN TRUSS, *T*-2, JACK TRUSS, *T*-1, AT RIGHT, SUPPORTS TWO CANTILEVERS.

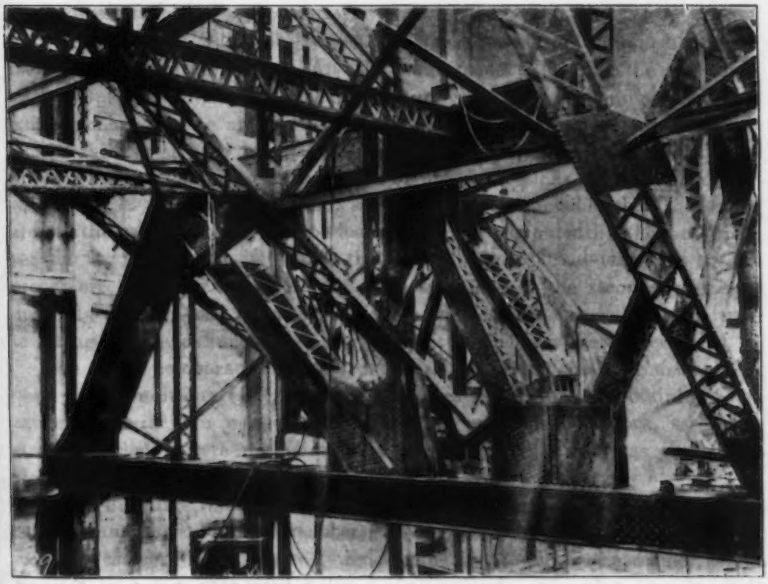


FIG. 8.—DETAIL OF MAIN BALCONY TRUSS, *T*-2, SHOWING PIN CONNECTION TO JACK TRUSS, *T*-1, AL MALAIKAH TEMPLE.

An adaptation of Mohr's semi-graphical analytical system, by Mr. G. A. Maney,* was selected by the writer for calculating the secondary stresses. Maney's arrangements of Mohr's methods are much simpler, more concise, and involve less time than other commonly accepted methods.

Only approximate results to determine probable localized excessive stress conditions were expected, on account of the modifying effects of various conditions, such as the following:

(1) Change in the primary stress distribution due to changes in the section of members (particularly at joints), to gussets and splice members, and also to fabrication conditions.
(2) Effects of changes in moments of inertia in members between joints.
(3) Variation between the theoretical length of a member between axial intersection points and the actual length between gussets.
(4) Elastic deformation of the gusset-plates, initial slip, and distortion of the rivets, and degree of frictional resistance between riveted plates.
(5) Unknown frictional resistance of pin joints.

In computing the deformations in the various members, the gross areas were used, no allowance being made for details or for changes in section at the connections. Mohr's successive approximations (to the third substitution) were utilized, which gave results accurate enough for all practical purposes. All computed secondary stresses were checked by another method of calculations to results within 5 per cent.

A maximum of 24 000 lb. per sq. in. was allowed for combined stresses, including secondary stresses. An effort was made to modify the secondary stresses due to dead load conditions by lengthening and shortening the various truss members in the amount of their respective deformations, which necessitated the use of hydraulic jacks, steamboat ratchets, and tapered drift-pins to force the members into their riveted or pin connections. As the dead load was about 72% of the total load, and as it was improbable that the entire area of the balcony would ever be loaded with the computed live load of 75 lb. per sq. ft., the seats being fixed, the writer deemed it advisable to try, by proper erection methods, to eliminate in some degree the secondary stresses due only to dead load conditions. While considerable force had to be used to spring the various members into place, little difficulty was experienced.

Due to the comparatively small deformations of the truss members under dead load conditions (Fig. 9), and the impracticability of obtaining such extreme accuracy in measurements in the structural shop work, secondary stresses probably were not eliminated to the desired extent, as subsequent strain-gauge measurements have indicated.

Some difficulty was encountered with hand-reaming. Due to large holes (1⅛ in.) and thick built-up members, it was difficult for the reamer crews to keep the reamers steady, there being a tendency to wobble the reamer, which resulted in a few over-sized holes.

* "Secondary Stresses and Other Problems in Rigid Frame," *Bulletin*, Univ. of Minnesota, March, 1925.

Balcony Truss Erection

Some doubt had been expressed regarding the ability and facilities for manufacturing the large truss near-by. Its successful fabrication and erection by a Los Angeles contractor (The Llewellyn Iron Works) was an achievement that created considerable local pride.

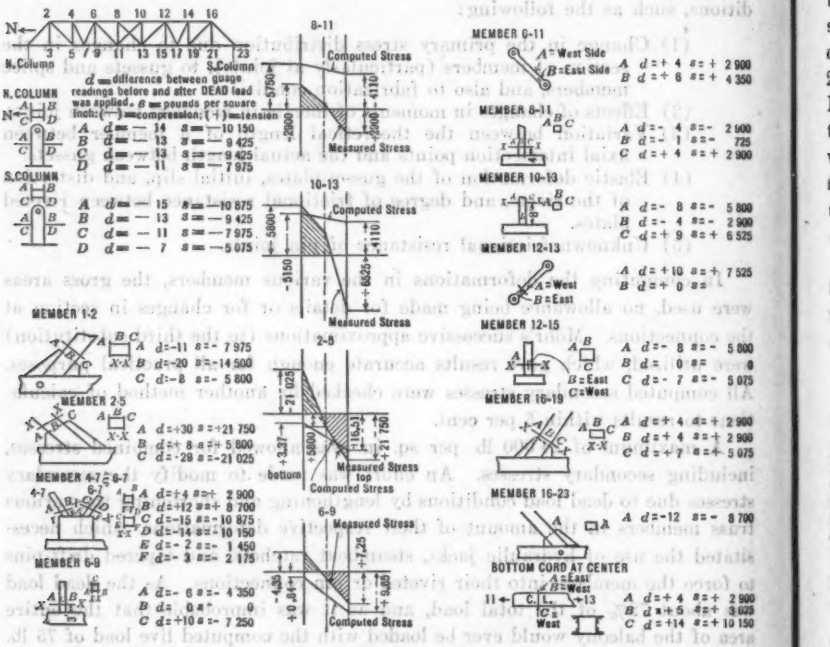


Fig. 9.—Strain-Gauge Investigations, Main Balcony Truss, T-2, Al Malaikah Temple.

This balcony truss was delivered to the job in sections, the largest (center lower chord section) weighing 47 tons and being 80 ft. long. The truss was erected on a steel substructure consisting of the same members as were used in the falsework for the roof trusses. The lower chord sections were first set on the camber screw-jacks. The web members were next placed, followed by the top chord sections.

No unusual difficulty was experienced in forcing the various members into their connections, and the pins were driven home without any undue effort. Prior to riveting, at least 30% of the holes in the connections were bolted tightly. No trouble was encountered in driving the 1-in. rivets (some with a maximum grip of 5¾ in.) by ordinary pneumatic hammers, with 120-lb. air pressure. Comparatively few rivets were rejected. The 1-in. rivets known as "swell-neck" rivets, were tapered a full ⅛ in. for 1 in. along the shank, starting at the head.

The taper on large rivets helped materially to fill the holes completely, particularly in over-reamed holes. In general, they resulted in fewer loose rivets, especially where the rivet grip was large.

The general layout of the balcony framing is shown in Fig. 10. The jack trusses, *T*-1, tend to equalize the over-hang of the cantilever trusses, which project a maximum of 45 ft. beyond the main balcony truss. It was desirable to prevent excessive and unequal deflections of the structural steel trusses supporting the balcony, particularly the cantilevers, and the possibilities of cracking and otherwise shearing the relatively thin monolithically finished 2½-in. slabs and narrow 5-in. risers of the reinforced concrete balcony framing. Hence, it became necessary to design and arrange the balcony trusses so that their deflections would be relatively uniform and slight. Deflections of the various trusses were carefully computed by Williot diagrams and checked analytically. The maximum computed deflection of the cantilever trusses under dead load was ¾ in. (Truss *B*).

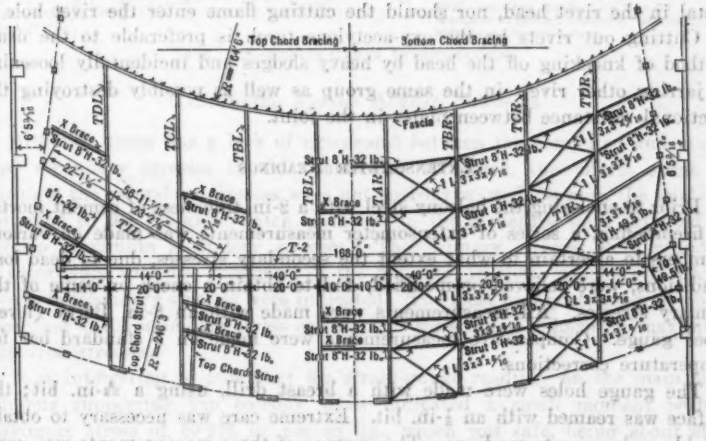


FIG. 10.—BRACING FOR BALCONY TRUSSES.

The cantilever balcony trusses were cambered for dead load by offsetting the rivet holes at the fulcrum connections in the gusset-plates on the main truss or the jack trusses, as the case may be, the top holes being set back and the bottom holes brought forward. The rivet holes in the gusset-plates at the fulcrum connecting the cantilever trusses to the main truss or jack trusses were drilled and reamed in the field after the cantilever trusses had been supported in their proper cambered positions by cables with turnbuckles secured to the roof structure. In adjusting the cantilever trusses to their proper camber the milled joint at the heel or fulcrum in the bottom chord was displaced so that the abutting faces in the joint were not in proper contact. This was corrected in a very satisfactory manner by filling the gap with electrically welded metal.

Electric Welding

Electric welding was found to be an excellent method for obtaining full metal-to-metal contact where milling of inclined surfaces cannot be made uniformly tight. The joint should be properly scarfed to allow the metal to be fluxed or built in to the inner side of the joint. Electric welding does not overheat, and possibly injure, the adjoining metal as will often result from the use of the oxy-acetylene torch in such operations.

Cutting Out Rivets

Where rivets had to be removed the oxy-acetylene torch was used to good advantage to burn off the button of the rivet. This operation, however, should be performed only by a mechanic experienced in using the torch, otherwise the metal in the joint may be injured. The cutting flame should be applied to the side of the rivet head at right angles to, rather than parallel with, the shank of the rivet. It is not always necessary to burn off all the metal in the rivet head, nor should the cutting flame enter the rivet hole.

Cutting out rivets by the oxy-acetylene torch is preferable to the usual method of knocking off the head by heavy sledges and incidentally loosening or jarring other rivets in the same group as well as possibly destroying the frictional resistance between plates in the joint.

Extensometer Readings

Prior to enclosing the balcony steel with a 2-in. thickness of cement mortar as fireproofing, a series of extensometer measurements was made on various members to ascertain to what extent the secondary stresses, due to dead load conditions, were relieved or modified, and to obtain a check on some of the primary stresses. All measurements were made with an 8-in. Berry (lever-type) gauge. Comparative measurements were made on a standard bar for temperature corrections.

The gauge holes were made with a breast drill, using a $\frac{3}{32}$-in. bit; the surface was reamed with an $\frac{1}{8}$-in. bit. Extreme care was necessary to obtain reliable and accurate readings. The average of three measurements was used. A slight difference in pressure on the points of the gauge when applying the extensometer produced considerable variation in the readings. The points on the gauge seemed to have too great a taper, being about 60 degrees. Points with sharper tapers, about 30°, and smaller gauge holes, properly reamed, doubtless, would give more accurate readings.

Readings to the number of 360 were taken on the balcony truss. Due to obstructions and inaccessibility, it was impossible to obtain complete extensometer readings on all truss members, and at many desirable points, to obtain the maximum secondary stresses. However, sufficient strain-gauge measurements were taken to demonstrate, in a practical way, what members were under maximum stresses. From the results obtained it appears that the stresses in the members varied considerably over the cross-sections, and that the maximum measured stress was well within safe limits.

The measurements indicated that the results of the field methods to modify the computed secondary stresses were rather uncertain, possibly due to relatively small deformations and inaccuracy in shopwork, and that the calculated secondary stresses did not fully agree with the measured stresses (Fig. 11). Doubtless this was due to the several modifying effects already mentioned, which are not subject to analytical calculations.

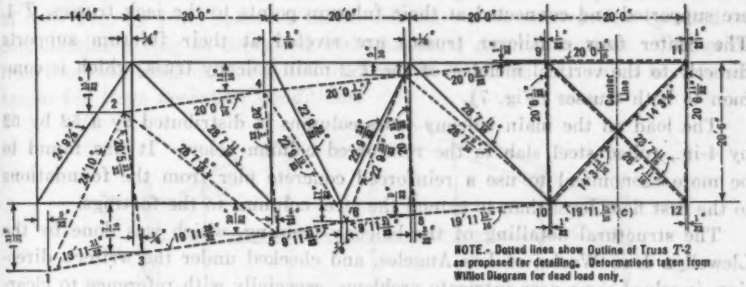


FIG. 11.—STRAIN-GAUGE INVESTIGATIONS.

CALCULATED *Versus* ACTUAL STRESSES

Although there was a lack of agreement between the computed and measured secondary stresses in this particular structure, the efforts spent to calculate the secondary stresses were not entirely wasted, because the analysis indicated to the designer which members in the structure were expected to develop maximum secondary stresses. The primary stresses were reduced, in accordance with the judgment of the designer, in those members in which excessive secondary stresses were indicated. The most highly stressed members according to the secondary stress calculations were found to have the maximum measured stresses.

It is interesting to note that the strain-gauge readings on the main steel columns supporting the balcony truss indicated a small moment with a maximum stress of 10 875 lb. per sq. in., which was safe, being about 30% greater than the computed stress. These columns were designed with a very low unit stress, 8 500 lb. per sq. in., to allow for increased stresses due to bending and other causes, which could not be computed accurately. Each of these columns is braced near its top by three 10-in. H-struts to the reinforced concrete wall structure. Note the variation in stress distribution on the section shown in Fig. 11.

BALCONY DESIGN

Truss *T*-1 is attached at one end to the main balcony truss by a pin connection in a large steel casting resting on a built-up stool between the web members of Truss *T*-2 (Fig. 8). The other end is supported on a reinforced concrete wall column. The chords of Truss *T*-1 consist of two 15-in. channels, with 8-in. Bethlehem H web members. The bottom chord

in the end panel section of the balcony truss, *T*-1, was composed of two 8-in. standard eye-bars to avoid clearance difficulties. The pin joint connecting Truss *T*-1 to the main balcony truss, *T*-2, eliminated any possibility of excessive restraining moments due to the varying deformations of the connected trusses.

There are eight cantilever trusses (Fig. 10), four of which are directly supported and connected on the main balcony truss, *T*-2; the other four are supported and connected at their fulcrum points to the jack trusses, *T*-1. The center four cantilever trusses are riveted at their fulcrum supports directly to the vertical member of the *T*-2 main balcony truss, which is common to both trusses (Fig. 7).

The load on the main balcony truss columns is distributed by a 52 by 52 by 4-in., rolled steel slab to the reinforced column below. It was found to be more economical to use a reinforced concrete pier from the foundations to the first floor level than to extend the steel columns to the footings.

The structural detailing of the balcony framing, which was done by the Llewellyn Iron Works, of Los Angeles, and checked under the writer's direction, involved some very intricate problems, especially with reference to clearances. That only two minor errors, due to changes in plans, occurred in the field erection is an accomplishment worthy of mention.

In order to stiffen the ends of the cantilever trusses and reduce their deflections to a minimum, a double web plate girder section was used, as shown in Fig. 7. During erection, the plate girder sections were given a camber by offsetting the rivet holes in the bottom chord in the joint connecting them to the open web section of the truss. The chords were built of 8 by 3-in. angles, latticed and placed 1 ft. 3 in., back to back, which width added to the lateral stiffness. The anchor, or wall, ends of the cantilever trusses were secured to anchor-bolts on top of reinforced columns.

Special attention was given to the design of the general bracing throughout the balcony framing. The cantilever trusses were sway-braced in horizontal and vertical planes by angle cross-bracing and struts in the form of bents, as indicated in Fig. 10. The cantilevers and jack trusses assisted materially in bracing the main balcony truss in addition to the four horizontal boxed latticed struts which braced the upper chord to the rear wall (Fig. 8). The four exits passing through the main truss were supported by hanger rods anchored above in the concrete deck of the balcony.

To avoid any possibility of the concrete balcony cracking along the Auditorium walls, due to deflections of the supporting trusses, a construction joint was placed in the concrete deck, midway between the Auditorium wall and the adjacent cantilever trusses. Although the balcony has been loaded to capacity several times, no cracks in the concrete decking have been observed to date.

The local building ordinance required a full live load test of the balcony, which was accomplished by using metal plaster barrels filled with water. After the test the water was siphoned from the barrels.

Unusual Stage Construction Features

The stage roof arches, eight in number, are spaced 22 ft. on centers. One end of each of the center arches is supported by the proscenium arch wall. The arches were designed as partly fixed (Fig. 12). Turnbuckles were placed in the bars near the west wall to develop equal stresses in the two channels that form the horizontal tie and to produce a small amount of initial stress in the tie prior to stripping the forms. The gridiron floor proper consisted of flats 4-in. by ⅜-in., spaced 7 in. on centers, which construction resulted in considerable saving as compared with the usual channel sections. The gridiron floor was designed for a live load of 100 lb. per sq. ft., which has proved ample for stage operating conditions.

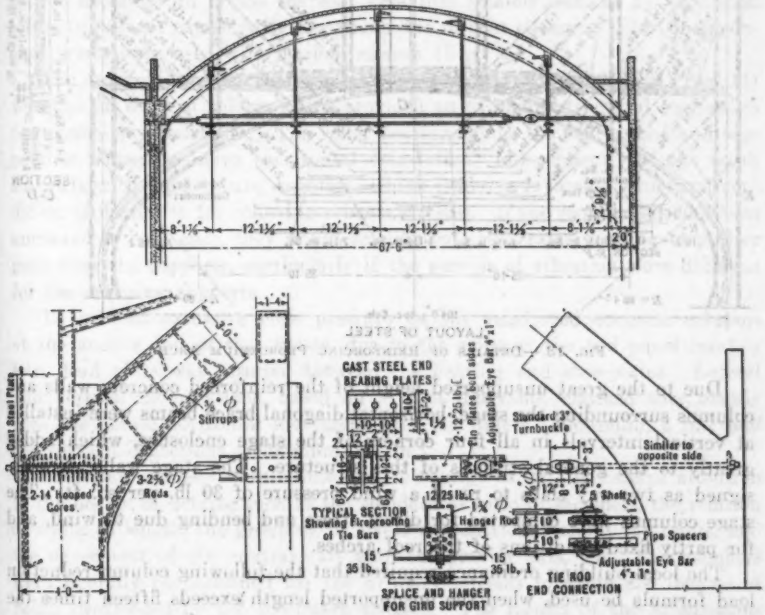


Fig. 12.—Stage Roof Arches.

The stage roof arches were poured in one continuous operation, commencing from each springing line and working simultaneously and symmetrically toward the crown. The arches and the beam system were poured up to the under side of the roof slab. By casting the arched roof slab with concrete having a slump of 2½ in., cover boards were found to be unnecessary. The roof arches were cast with a concrete having the proportions of 1 part cement to 4.5 parts aggregate.

The proscenium arch (Fig. 13), has a clear span of 100 ft. This arch was computed as a girder with partly restrained ends. Particular attention was given to diagonal tension stresses, which were resisted by diagonal bars

(Fig. 13). In order to increase the lateral stiffness, pilasters were placed in the web under each roof arch similar to the stiffeners in plate girder construction. A flanged head, or tee, in the top of the proscenium girder also adds to its lateral stiffness.

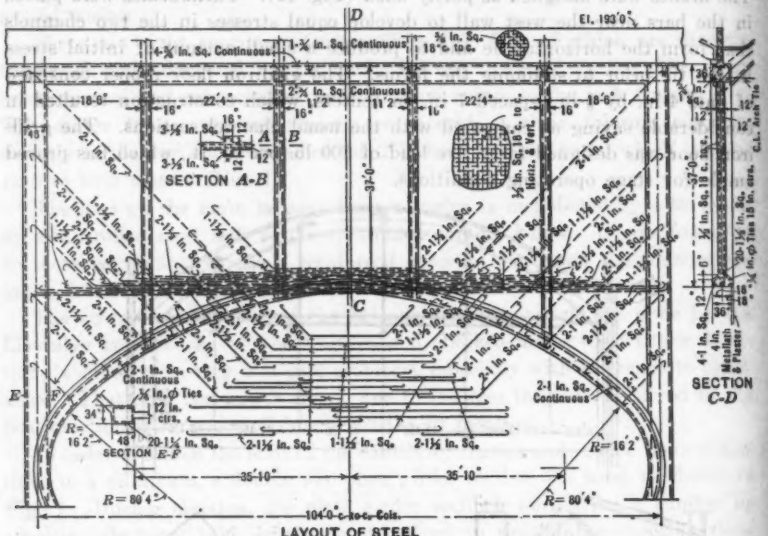


FIG. 13.—DETAILS OF REINFORCING PROSCENIUM ARCH.

Due to the great unsupported height of the reinforced concrete walls and columns surrounding the stage, horizontal diagonal brace beams were installed at vertical intervals in all four corners of the stage enclosure, which added greatly to the general stiffness of this structure. The stage walls were designed as two-way slabs to resist a wind pressure of 30 lb. per sq. ft. The stage columns were computed for direct stress and bending due to wind, and for partly fixed conditions of the roof arches.

The local building ordinance required that the following column reduction load formula be used, when the unsupported length exceeds fifteen times the least diameter:

$$P' = P\left(1.6 - \frac{L}{25\,D}\right)$$

in which,

P' = permissible load.
P = permissible load due to ordinary column formula.
L = unsupported length.
D = least dimension of core.

The stage floor was designed for a live load of 250 lb. per sq. ft. The framing consists of standard steel I-beams supporting a wood-joist system. All steel connections were bolted so that any particular section of the floor could be removed readily without affecting the adjacent parts.

Auditorium Roof Trusses

The structural steel roof trusses over the Auditorium (Fig. 14), each weighing 60 tons, have a clear span of 192 ft. It will be noted that these trusses are supported on reinforced concrete columns.

The Auditorium proper was constructed with double walls on each side to resist earthquake forces and provide duct space for the ventilating systems. The inner walls of the Auditorium are metal lath and plaster, supported by reinforced concrete columns and beams. This skeleton frame forms a system of braced bents with the outer walls and columns which support the roof trusses (Fig. 15).

Due to the large area of the Auditorium and the long-span roof trusses, it was necessary to divide the roof area into smaller sections by expansion joints to prevent excessive deformations due to heat changes. Elastite expansion joints were used with marked success (Fig. 15).

The south ends of the roof trusses were anchored; the north ends (Fig. 14) were set on nests of rollers. This provided an excellent structural connection for earthquake resistance. The roller bearings allow the center roof and supporting trusses to move (horizontal translation) independently of the north supports with temperature variation, which is also a favorable structural condition of flexibility for relieving seismic stresses. If the roof trusses had been anchored at both ends, they would act like battering rams, tending to pull or push over the supports, particularly if the periods of vibration were different for the opposing supports.

To prevent excessive edge pressure on the reinforced concrete columns at the anchor end of the trusses, due to the slope at the end panel joint, a $\frac{1}{8}$-in. lead plate was inserted between the bearing and shoe-plates. Lateral and vertical deformations, based on Williot diagrams, checked very closely with the actual horizontal movement over the rollers (maximum, $\frac{3}{4}$ in.) and the vertical deflections (maximum, $2\frac{1}{16}$ in.).

Secondary stresses were eliminated, to a large extent, by careful attention to the design of details, such as the slenderness ratio of members, the relation of width to length, the grouping of rivets concentrically about the joints, and the agreement of the neutral axis with the gravity axis, etc. The members were liberally proportioned for the primary stresses to avoid the danger of being overloaded by the secondary stresses. The results of the calculations indicated that the combined primary and secondary stresses did not exceed 21 000 lb. per sq. in. Because of the relative low secondary stresses, it was not deemed necessary to attempt to eliminate them by shop or erection methods.

The steel roof trusses were erected in pairs from a movable substructure (Fig. 14), consisting of braced steel bents bolted together and resting on rollers over timber cribbing. After all the roof trusses were erected, the movable steel substructure was dismantled and its members were used in the temporary supports for erecting the main balcony trusses.

The roof trusses were first assembled in the shop to the proper cambered positions, reamed, and match-marked. The trusses were erected by sections. The bottom chord was assembled first on the camber blocks of the movable

substructure, then the web members were installed, followed by the top chord sections. All members were bolted tightly with at least 30% of the rivet holes in the joints bolted. A great saving in the roof steel (about 14 tons) was effected by using a cantilevered purlin system, with alternate double cantilever and suspended members.

The computations relative to cantilever purlins are based on full uniform load conditions, in which case the most economical cantilever projection is 0.1465 of the distance between the trusses. For partial loading, the maximum moment due to uniform loads would be increased 20 per cent.

Pavilion Roof Girders

The continuous reinforced concrete girders, supporting the Banquet Hall roof, were designed as a part of a fixed frame (Fig. 16). The soffits of the roof girders (Fig. 4) were curved to represent Saracenic arches, which added to the depth of the girders over the interior columns where large negative moments existed, and presented a very pleasing architectural appearance. It will be noted that the balcony cantilever girders tended to reduce the moments in the columns. The rear, or wall, columns were required to resist an uplift. The anchor arms of the continuous girders were projected through and above the roof (Fig. 16), being hidden from the interior.

Foundations

The footings were located in a tight gravel stratum. A maximum of 8 000 lb. per sq. ft. was adopted as the safe bearing pressure. However, this maximum pressure was modified. The foundation having the largest ratio of live, to dead, load was selected and proportioned for the combined dead and live loads. The dead load on this footing was divided by the area thus formed, and the new reduced unit pressure was used for the dead loads on the foundations.

Considerable economy was obtained by combining the exterior retaining walls with the continuous footings, particularly under the east exterior wall of the stage. The area of steel for resisting the tension stresses, due to large moments in the footings, was reduced to a great extent, as the advantage of a relatively much larger moment arm was obtained by locating the positive moment steel in the upper part of the retaining wall. This is practically monolithic with the footing. The spread, or T-shape, of the footing provided adequate compression area.

Construction Plant

The contractor's construction plant and system of operation were of unusual interest. The concrete had to be distributed over an area of about 3 acres, which necessitated a gravity distributing system and central mixing plant. The distributing tower and concrete plant were located in the center of the stage, and the main distributing tower, a few feet in front of the proscenium arch.

Various reasons influenced the selection of the stage as the most practical location for the distributing tower and concrete plant. The mixer was centralized, which, to a large extent, equalized distribution of the concrete. The

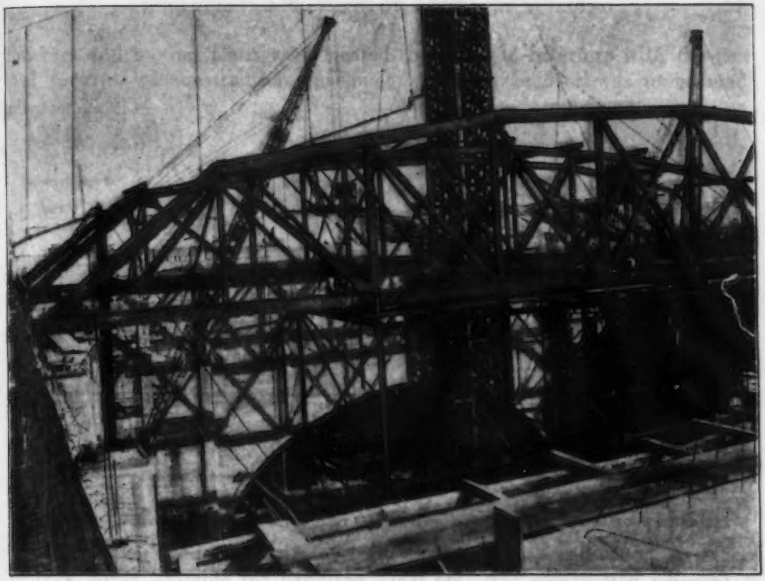


FIG. 14.—ROOF TRUSSES, ROLLER END SHOWING MOVABLE STEEL FALSEWORK, AL MALAIKAH TEMPLE.

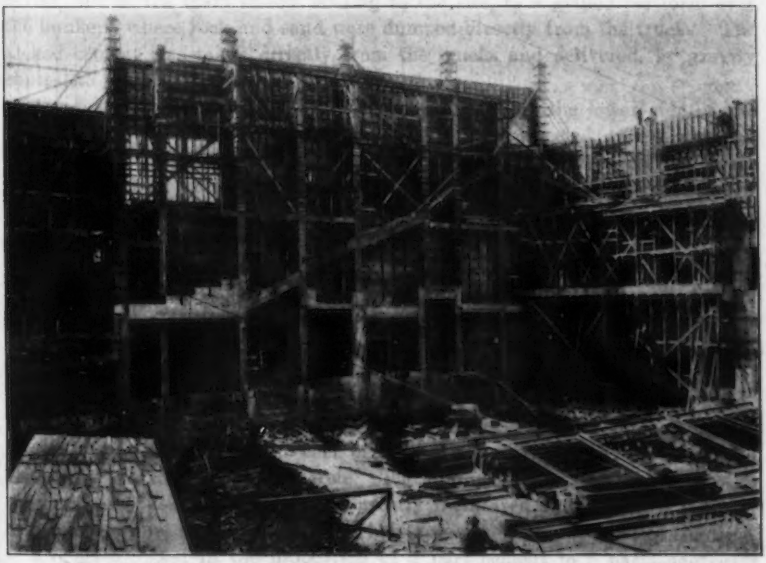
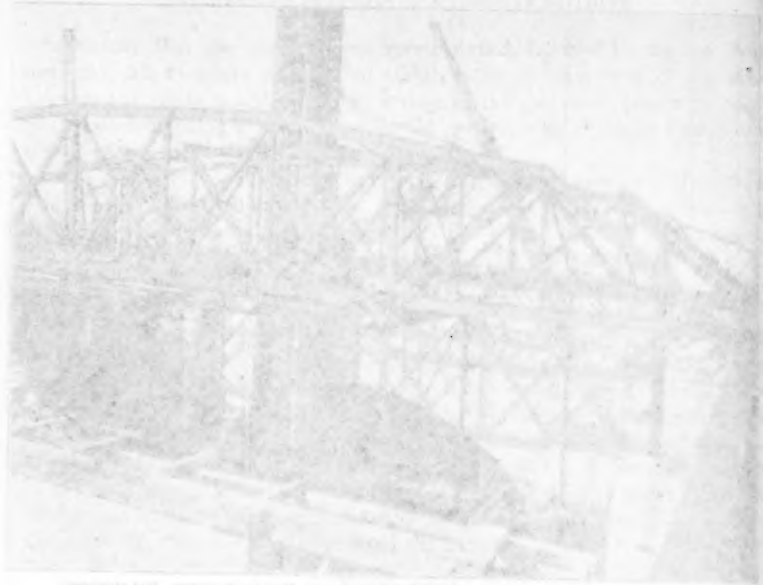


FIG. 15.—VIEW OF INNER WALL FRAMING AND SOLID EXTERIOR SIDE-WALL, AL MALAIKAH TEMPLE.



bunkers and mixing plant were located so as not to interfere with the general progress of construction, inasmuch as the stage floor was to be made removable.

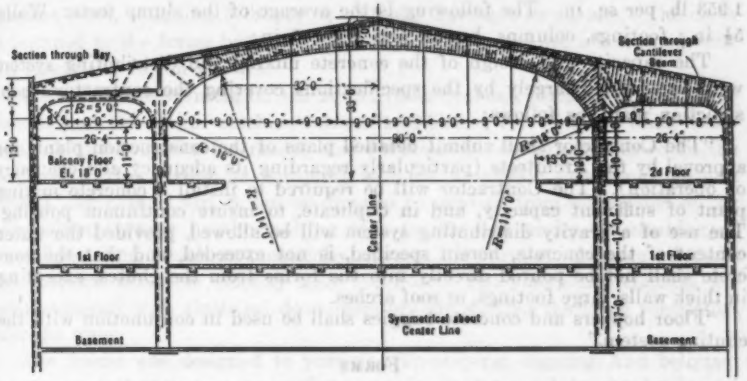


FIG. 16.—CROSS-SECTION OF BANQUET HALL, SHOWING CONTINUOUS ROOF GIRDERS.

The bunkers had a capacity of 1 000 cu. yd. of rock and sand. A cement storage house, suitable for handling 3 000 bbl., was constructed adjacent to the plant. For truck delivery of materials a ramp was built from Jefferson Street, one of the main streets leading to the site, to a grilled platform over the bunkers, where rock and sand were dumped directly from the trucks. The sacked cement was taken directly from the trucks and delivered, by gravity chutes, to the storehouse.

Sand and gravel were fed from short spouts along the lower side of the bunkers into regular concrete buggies, which were wheeled an average distance of 20 ft. to the mixer hoppers, and dumped. The concrete aggregate was measured by volume in the concrete buggies. It required three buggy men to feed each mixer when operating at full capacity. The mixing water was controlled by an automatic water tank.

To prevent interruption, due to possible break-downs in the construction plant while casting important structural units for which continuous pouring was necessary, a duplicate plant was specified and installed. This consisted of a two-compartment hoisting tower to which was attached two 1-yd. sliding distributing hoppers and two 1-cu. yd. Smith mixers. The main distributing tower was 238 ft. high. Two tail towers, each 100 ft. high, were built about 300 ft. each from the main hoisting tower; a 1½-in. plow steel cable, 1 500 ft. long, was laid over the hoisting tower to each of the tail towers to support the Lakewood chutes for distributing the concrete. The duplicate plant, in addition to its protection in case of an emergency, greatly facilitated and expedited the general construction, as both mixers and hoists were kept in use most of the time.

Concrete mixed in the proportion of 1 part cement to 6 parts aggregate was used throughout, except in the stage roof arches, columns, and roof arches of the Banquet Hall. All roof arches were cast with a 1:4.5 concrete, and the columns supporting the roof arches of the Banquet Hall were poured with a 1:3 mix.

The strength of the concrete was controlled by mechanical analysis and by slump and compression tests. For the 7-day compression tests, 1:6 mix, the average of 119 tests was 1 155 lb. per sq. in.; and for 190 tests at 28 days, 1 953 lb. per sq. in. The following is the average of the slump tests: Walls, 5½ in.; footings, columns, beams, and slabs, 5 in.

The capacity and design of the concrete mixing and distributing system were determined largely by the specifications covering the contractor's construction plant, as follows:

"The Contractor shall submit detailed plans of the construction plant, for approval by the Architects (particularly regarding its adequacy and efficiency of operation). The Contractor will be required to install a concrete mixing plant of sufficient capacity, and in duplicate, to insure continuous pouring. The use of a gravity distributing system will be allowed, provided the water content of the concrete, herein specified, is not exceeded, and that the concrete shall not be poured directly into the forms from the chutes, excepting in thick walls, large footings, or roof arches.

"Floor hoppers and concrete buggies shall be used in conjunction with the chuting system."

Forms

The specifications covering the design and erection of form work, contained several paragraphs which represented considerable thought and experience. These specifications read in part:

"In proportioning forms and centering concrete shall be treated as a liquid of its full weight for vertical loads and three-fourths of its weight for horizontal pressures.

* * * *

"Timber supports must be designed so that the number of joints in side-grain compression shall be reduced to a minimum on account of the low bearing value of timber across the grain.

"Care must be taken to prevent lateral displacement of vertical posts due to radial pressures from arch rings, by providing proper longitudinal bracing.

"Vertical supporting members, posts, struts, etc., must have butt-joints or end grain bearing and fish plates. Fish plates shall not be permitted to transmit vertical loads. Scabbing of horizontal supporting members to vertical supports will not be permitted.

* * * *

"An additional rise in the centering of arches of $\frac{1}{800}$ of the span shall be provided for shrinkage and settlement of the forms and supports.

"In constructing centers for arches no reliance shall be placed on spiking, but all main members shall be bolted together at joints.

* * * *

"The design and construction of all form work shall be subject to the approval of the Engineer, and no concrete shall be deposited on or in said form work prior to such approval; but such approval shall not be deemed cause for claims for damages or extension of time in case of failure of said form work, nor will it relieve the Contractor from furnishing forms which will produce acceptable work. All such approved plans and descriptions of said form work shall thereupon become a part and parcel of these plans and specifications."

Fireproof Curtain

The asbestos stage curtain is one of the largest in the world, weighing approximately 9 tons. It consists of a rigid steel frame composed of six hori-

zontal latticed trusses 14 in. deep at the center and braced vertically by latticed angle struts extending the entire height of the curtain.

The steel curtain frame is covered on each side with wire-woven, 95% pure, asbestos cloth, weighing 3 lb. per sq. ft.* and having wire inserts. The cloth is fastened to the frame by half round strips, which, in turn, are secured with tap screws, 12 in. on centers.

The curtain is operated on ten 20-in., roller-bearing, machine-turned, cable sheaves and two 24-in. machine-turned head-blocks, set on large roller bearings. A 10-h.p. motor for raising and lowering the curtain is located in the basement. The curtain is equipped with an automatic emergency closure control. In case of fire, it can be dropped by gravity from either side of the proscenium opening by the melting of a fusible link or by the slashing of a small cut line, which automatically releases the curtain. A large oil dashpot prevents excessive velocity near the floor. The emergency controls are interconnected with electrical door operators for opening the large ventilators over the stage.

The frame was designed to prevent side-swaying, sagging, and bellying, due to unequal air pressure on the two sides, and to withstand a horizontal lateral pressure of 5 lb. per ft., with a factor of safety of 2, without binding on the smoke-pocket guides. The lateral pressure is based on a temperature of 750° Fahr. on the Auditorium side and 1 600° Fahr. on the stage side, with 15-min. exposure.

The curtain has an unbalanced weight sufficient to cause it to drop by gravity with a frictional factor of 1 in 32 sec. As designed, the whole curtain descends as a unit, making its entire unbalanced weight effective in dropping.

The stage scenery equipment (Peter Clark System) consisted of 100 counterweighted sets, operated on T-bar tracks, about 4 in. on centers. A bridge located along, and cantilevered from, the rear wall of the stage provides the facilities necessary for painting the exceptionally large scenes.

The Acoustics

The acoustics of the Auditorium was the subject of much thought. Detailed study was given the correlation of size and shape to prevent objectionable reverberation. The selection of lime plaster for finishing the interior wall surfaces assisted, to some extent, in absorbing the sound. The great hung canopy and the surrounding ceiling, with its numerous broken surfaces, were treated so as to subdue undesirable reflection without losing any of its architectural beauty. All the seats were upholstered and the aisles carpeted. Not only was this in keeping with the luxuriousness of the appointments, but it also served further to prevent acoustical defects.

As a result, normally, stage programs can be heard without difficulty in the rear seats of the balcony, a distance of 235 ft. However, this was considered too great a distance for the average human voice, or subdued music, to carry successfully. Consequently, it was deemed advisable to provide an amplifying system.

* Same as tested by the U. S. Bureau of Standards, Washington, D. C., in its curtain tests.

Public Address System

In keeping with the high standards utilized throughout this edifice, it was desired to obtain the most comprehensive and complete system available, as well as one that would faithfully amplify, transmit, and reproduce the program. To meet these requirements, a Public Address System was installed by the Graybar Electric Company.

It is used for many and varied functions. In addition to reproducing speech and music from the stage of the Auditorium, it provides a means for "piping" the program from one room to another, thus caring for overflow meetings. This Public Address System is also used for amplifying in the large ballroom, for reproducing radio programs when used in conjunction with a radio receiving set, for paging throughout the building, and for cab calls.

To provide these features, it was necessary to install in and about the Temple a total of 46 projectors or horns, 25 microphone outlets, and 24 observer stations. A special control room, adjacent to the Exhibition Hall, was used for housing the amplifying and radio receiving equipments and auxiliary apparatus. The microphone outlets were installed at strategic points throughout the building for conveying, to the amplifying system, such programs as it is desired to reproduce throughout the building itself, or to transmit over the air through a radio broadcasting station.

Similarly, observer positions were provided for watching the performance of the loud speaker equipment in each particular room. A telephone circuit, between these positions and the control room, enables the observer to advise the operator if corrections are required. The power used for operating this system is obtained from the regular alternating-current house supply and is adjusted to the proper voltage by a vacuum tube rectifier system.

Since the Auditorium is thus equipped, it is not necessary to request front seats in order to enjoy the program. There are no panels to hinder the view, the projectors being carefully concealed in the proscenium arch 42 ft. above the stage. They reproduce the program so that those in the rear seats can hear as well as those in front.

Heating and Ventilating

The heating system consists of three Birchfield welded, low-pressure boilers, set in battery with Fess oil burners. Each boiler has a capacity of 30 000 sq. ft. of radiation. The Auditorium and Banquet Hall are heated and ventilated by a forced-air circulating system.

The majority of people who visit this edifice are so impressed with the lavishness of its appointments, that they are apt to take for granted the perfect air conditions that prevail without realizing the great engineering skill that made such conditions possible. The average layman would be astounded to know that, in order to insure perfect ventilation, it is necessary to handle 43 254 000 cu. ft. of air per hour. This enormous quantity is distributed by 12 Sturtevant ventilating fans, of various sizes, located at strategic points throughout the building. To the Auditorium alone 14 838 000 cu. ft. is supplied, and correspondingly large quantities are furnished to dressing-rooms under the stage, the Banquet Hall, basement, etc.

The Auditorium is ventilated by the up-draft system (Fig. 17), wherein the air is forced into plenum chambers under the seats and thence through mushroom outlets into the room. An exhaust air system, with outlets in the ceiling under the balcony and in the ceiling of the Auditorium, draws the vitiated air up past the breathing zone and exhausts it to the outside. The up-draft system for a building of this type is much superior to the so-called down-draft system, wherein the air enters the ceiling and is sucked out under the seats. In the up-draft system the laws of Nature are utilized as the heated air rises, which assists the exhaust system in its work. A very important point in favor of the up-draft system is the quick elimination of smoke, especially during the Shrine ceremonials.

The incoming fresh air is heated by large units of Aerofin heaters which are automatically controlled by thermostats set in the various rooms. The stage is amply heated to compensate for differences in temperature between the Auditorium and the stage area, thus eliminating the great drafts that usually occur when the curtain is raised.

Decoration

The architectural treatment of the entire structure is Saracenic, with the Moorish motif predominating. Rare beauty marks the ceiling of the Auditorium (Fig. 2) with its jeweled tapestry canopy reflecting the blue of an Arabian sky. The tapestry effect on the huge plastered canopy with its large ropes (9 in. in diameter) of cast plaster, is so realistic that many have believed that this immense ornament was constructed of real tapestry and fibered ropes.

The whole effect of the interior of the Auditorium is heightened and magnified by a gloriously beautiful envelope of Oriental filigree work and daring color schemes (Fig. 5), which is truly Saracenic in design. The Saracens were celebrated for their decorative art, in which the Oriental love for display and color demanded a certain luxuriousness, that is reflected through the interior treatment.

A great deal of decoration was applied directly to the concrete beam and wall surfaces, which were tinted and stenciled, producing a very striking and interesting effect, particularly on the roof arches in the Pavilion (Fig. 4).

The concrete exterior of the entire building was treated with California stucco of a light buff color, applied with an air brush and producing a durable coating of a pleasing uniform texture. The exterior ornamentation consists of polychromed tooled synthetic limestone, composed of crushed limestone and Portland cement.

Electric Lighting

In the center of the suspended canopy is hung a magnificent chandelier, 22 ft. high and weighing approximately 5 tons (Fig. 2). It contains 500 electric bulbs, arranged to produce 64 color combinations, all controlled by dimmers on the stage switchboard. There are fifty-one 1 320-watt circuits, nine 150-watt lamps to each circuit, and three emergency circuits. To each of the four colors—red, white, blue, and amber—there are twelve circuits.

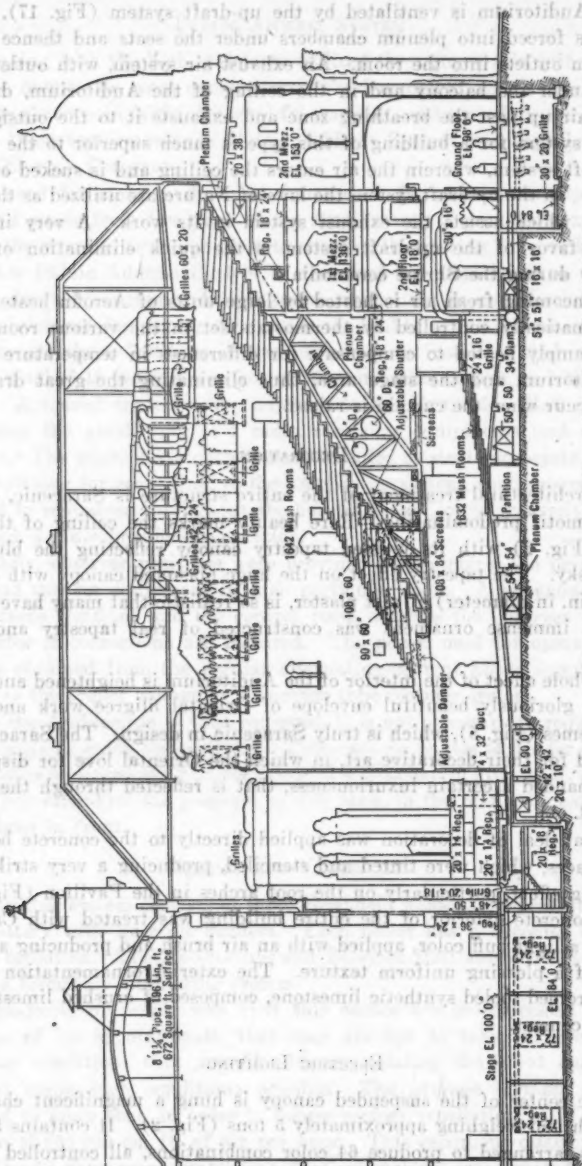


FIG. 17.—LONGITUDINAL SECTION OF AUDITORIUM, SHOWING UP-DRAFT SYSTEM OF VENTILATION.

The chandelier is constructed with a structural steel frame connected to the steel ceiling supports by pin joints, to allow the fixtures to swing during seismic disturbances. Access to the fixture for lamping and cleaning is provided by a portable ladder extending through a trap-door in the ceiling.

The entire lighting system is unique. Hundreds of concealed lights of varying colors, together with the large chandelier and other minor fixtures and all the stage lighting equipment, are controlled by one of the largest stage switchboards in the world. This switchboard, manufactured by the Hub Electric Company, is 28 ft. long and 12 ft. high, weighs 16½ tons and contains 298 dimmer plates of the Ward Leonard selective remote control. The dimmers run the full length of the board. By a system of roller-bearing mountings, a full length of dimmer can be controlled by a master lever.

Two separate electric services enter the building, connecting to two separate transformer vaults. A 100-kw. transformer set is installed for direct current for use on the stage.

Miscellaneous

A total of 30 275 cu. yd. of concrete were poured, and 2 308 000 board-ft. of form lumber was utilized. About 1 550 tons of reinforcing steel and 1 100 tons of structural steel were used.

The following shows the amount of each contract, and the total cost of the building:

Excavating	$38 520.00
General construction	1 214 452.00
Plumbing, heating, ventilating, and sprinklers	190 000.00
Electric wiring	109 532.00
Switchboard	41 939.00
Elevators	21 570.00
Electric fixtures	38 641.00
Decorating and painting	68 000.00
Fixed seating	50 000.00
Stage equipment	60 984.00
Finish hardware	13 140.00
Public Address System	16 246.00
Organ	35 000.00
Sign lights	1 295.00
Estimated cost of store fronts	8 000.00
Estimated cost of kitchen equipment	8 500.00
Total	$1 915 819.00
Architects' fees and miscellaneous small items	140 000.00
Total cost of building alone	$2 055 819.00

Mr. John C. Austin was the Architect, and Edwards, Wildey and Dixon, the General Contractors. Thomas Haverty Company furnished the heating, ventilating, and plumbing systems. The writer, as Chief Engineer for the Architect, was responsible for the structural design and the design of the heating and ventilating systems.

DISCUSSION

THEODORE BELZNER,* AFFILIATE, AM. SOC. C. E. (by letter).—The author states that "a slight difference in pressure on the points of the gauge when applying the extensometer produced considerable variation in the readings". During 1913 and 1914, a comprehensive series of extensometer investigations was made on the important truss members at the main towers and legs of the Brooklyn intermediate towers, during the strengthening of the end spans of the Williamsburgh Bridge, in New York City.

A 20-in. Howard extensometer, with 55° tapered points, was used, and the writer does not recall such variations as the author indicates, except in cases where the pressure on the strain-gauge was not properly applied. When the pressure was always placed directly over the contact points and firmly held against the member to be measured, there could not possibly be any spring of the various parts in the strain-gauge, causing variations in the readings.

The gauge holes in the steel members were drilled with a hand ratchet, using a No. 57 combination drill bit with a 60° countersink. After the holes were drilled, the edge between the countersink and drilled holes was removed with a 55° center punch by tapping it with a hammer which, in turn, formed a conical seat to receive the points of the strain-gauge. A pointed aluminum rod, a small cedar stick, and some absorbent cotton were used for cleaning the holes before taking any readings. By using the aluminum rod, the softness of the material prevented any injury to the holes. The gauge holes were cleaned out at each reading.

The writer agrees with the statement concerning the reamed holes, and believes that with careful and accurate preparation, the application of proper pressure on the strain-gauge, and the extreme care taken by the author should have resulted in more accurate readings with the equipment used on the theatre building.

Detailed descriptions of precise extensometer investigations made on the Hell Gate Arch Bridge† and the writer's discussion of the Niagara Railway Arch Bridge,‡ may be of interest in connection with this paper.

FRED A. NOETZLI,§ M. AM. SOC. C. E. (by letter).—The design of the theatre building described by Mr. Beanfield involved some very unusual features. Especially noteworthy are the balcony truss of 168-ft. span, the balcony cantilevers with a 45-ft. overhang, the reinforced concrete proscenium arch of 100-ft. span, and the reinforced concrete frames over the Banquet Hall, having a center span of 90 ft.

The writer had the privilege of being connected with the design of some of the structural parts of the building, particularly those of reinforced concrete. In the original plans there were provided for the Auditorium roof, two hinged reinforced concrete arches with horizontal ties spanned between the arch abutments. This type of construction would have involved some espe-

* Insp. of Steel and Bridge Insp. in Chg., Brooklyn Bridge, Dept. of Plant and Structures, City of New York, Brooklyn, N. Y.

† *Transactions*, Am. Soc. C. E., Vol. LXXXII (1918), pp. 1047–1051.

‡ *Loc. cit.*, Vol. LXXXIII (1919–20), pp. 2004–2005.

§ Cons. Hydr. Engr., Los Angeles, Calif.

cially interesting features, but steel construction was subsequently used, as described in detail by the author.

The proscenium arch (Fig. 13) is supporting, among other loads, the concentrated vertical reactions of four of the stage roof arches. Considerations from the architectural point of view prohibited the use of a horizontal tie to take the large thrust of this arch. On the other hand, the columns supporting the arch could not economically be constructed with sufficient lateral stiffness and strength for carrying this thrust. Consequently, the most economical solution appeared to be the design of this "arch" as a girder and for this purpose the great depth available for the girder (37 ft. at the center and about 60 ft. at the supports) was very desirable. Nevertheless, there were twenty 1½-in. bars necessary for the bottom reinforcement at the center of the span. The diagonal tension was taken care of by a combination of bent-up bars and stirrups.

The author's computation of the secondary stresses in the steel balcony truss are of special interest. These studies indicated a way of eliminating part of the theoretical secondary stresses by proper erection methods. Such a procedure has been used very successfully also for other types of structures, especially bridge arches in which the effect of rib-shortening was partly compensated for by putting the arches under initial compression by means of screw-jacks and, subsequently, effecting the closure of the arches.

Mr. Beanfield's efforts to check, with extensometer readings, the theoretical stresses in important structural members is commendable. Information on the size of secondary stresses in such structures is especially desirable. It is to be hoped that such strain measurements will also be made to serve as a guide in the design of future structures.

A. Floris,* Esq. (by letter).—The most interesting part of this structure is doubtless the Banquet Hall. Unfortunately, however, the author's statements with regard to this important part of the design are lacking in detail to such an extent that it is difficult to understand clearly the intentions of the designer.

The structure (as shown in Fig. 16), represents a rigid frame which, due to the absence of hinges or other design features eliminating bending moments, is statically indeterminate to a high degree. Obviously, to analyze such a complicated structure is practically not feasible and for this reason some simplifications must be introduced.

In such cases it is always possible, by a proper arrangement of hinges and other features of design, to divide the structure into secondary frames, beams, and columns the analysis of which is an easy matter. The structure (Fig. 18) may be divided into a two-hinged frame that supports the beams from the outside columns as well as the simple beam within the middle span. Also, it could be assumed to consist of two superimposed frames with two hinges each. The upper frame, with a tie-rod connecting two hinges, would be similar to that suggested first and the lower frame would be composed of the two columns and the straight beam within the middle span. Formulas for the solution of these frames are available elsewhere.†

* Los Angeles, Calif.

† "Rahmenformeln," by A. Kleinlogel, Berlin, 1925.

The solution of the latter subdivision results in reducing the bending moment of the beam in the middle of the lower frame. The same result can be obtained if this beam (Fig. 18) is fixed to the columns of the frame. In order to reduce the statically indeterminate quantities in such a case, a hinge is inserted in the top of the roof (Fig. 19) which simplifies the calculations appreciably.

FIG. 18.

If the loading is symmetrical, the system thus formed is statically indeterminate in the second degree. This assumption as to the loading corresponds fairly well to the actual conditions with the exception, perhaps, of the wind pressure acting on the frame. In this building, however, the influence of wind seems to be negligible.

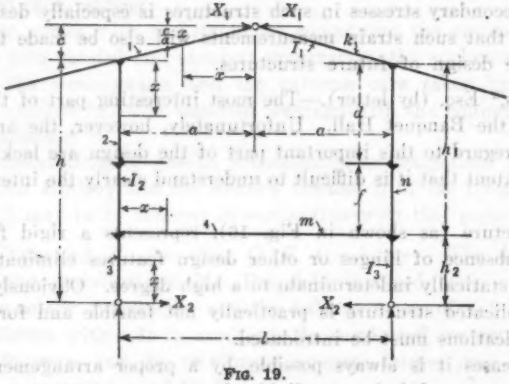


FIG. 19.

In the case of a symmetrical loading, the horizontal thrusts, X_1 and X_2 (Fig. 19), are taken as the unknown quantities. Then, if only the work done by the bending moments is considered, the equations of least work* are:

$$\frac{\partial u}{\partial X_1} = \int \frac{M}{E I} \frac{\partial M}{\partial X_1} d s = 0 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

$$\frac{\partial u}{\partial X_2} = \int \frac{M}{E I} \frac{\partial M}{\partial X_2} d s = 0 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

* "Die neueren Methoden der Festigkeitslehre und der Statik der Baukonstruktionen," H. Müller-Breslau, Leipzig, 1924, p. 96; see, also, "Mechanics of Internal Work," I. P. Church, N. Y., 1910, p. 63.

Denoting by M_0 the bending moment of the statically determinate frame for a given loading, the following moments and their derivatives for the various elements of the frame are obtained (see Fig. 19):

Element 1:

$$M = M_0 - X_1 \frac{c}{a} x; \frac{\partial M}{\partial X_1} = -\frac{c}{a} x; \frac{\partial M}{\partial X_2} = 0$$

Element 2:

$$M = M_0 - X_1 (c + x); \frac{\partial M}{\partial X_1} = -(c + x); \frac{\partial M}{\partial X_2} = 0$$

Element 3:

$$M = M_0 - X_2 x; \frac{\partial M}{\partial X_1} = 0; \frac{\partial M}{\partial X_2} = -x$$

Element 4:

$$M = M_0 - X_1 (c + h_1) - X_2 h_2; \frac{\partial M}{\partial X_1} = -(c + h_1); \frac{\partial M}{\partial X_2} = -h_2$$

Using these expressions, Equations (1) and (2) take the final form:

$$X_1 \left[2 \left\{ k s \frac{c^2}{a^3} \int_0^a x^2 \, dx + n \int_0^{h_1} (c + x)^2 \, dx \right\} + m (c + h_1)^2 \int_0^l dx \right] + X_2 m h_2 (c + h_1) \int_0^l dx = \alpha$$

$$X_1 m (c + h_1) h_2 \int_0^l dx + X_2 \left[2 \int_0^{h_2} x^2 \, dx + m h_2^2 \int_0^l dx \right] = \Gamma$$

or,

$$X_1 \left[2 \left\{ k s \frac{c^2}{3} + n c h_1 \left(c + h_1 + \frac{h_1^2}{3c} \right) \right\} + m (c + h_1)^2 l \right] + X_2 m h_2 (c + h_1) l = \alpha \quad \ldots\ldots(3)$$

and,

$$X_1 m h_2 (c + h_1) l + X_2 h_2^2 \left(\frac{2}{3} h_2 + m l \right) = \Gamma \quad \ldots\ldots(4)$$

in which,

$$s = \sqrt{a^2 + c^2}$$

$$\alpha = 2 \left[k s \frac{c}{a^2} \int_0^a M_0 x \, dx + n \int_0^{h_1} M_0 (c + x) \, dx \right] + m (c + h_1) \int_0^l M_0 \, dx$$

$$\Gamma = 2 \int_0^{h_2} M_0 x \, dx + m h_2 \int_0^l M_0 \, dx$$

and,

$$k = \frac{I_3}{I_1}; n = \frac{I_3}{I_2}; \text{ and } m = \frac{I_3}{I_4}$$

The frame was made statically determinate by removing the roof hinge (without connecting the separated parts) and also putting the right column on rollers (Fig. 20).

The integrals in Equations (3) and (4) depend on the loading and must be determined in each case separately. The final results are (Fig. 20):

Loading (*a*).—For a unit load, p, uniformly distributed over the entire span of the roof (cantilevers excluded),

$$\alpha = \frac{p\,a^2}{4}\left[k\,s\,c + 2\left\{n\,h_1\,(2\,c + h_1) + m\,l\,(c + h_1)\right\}\right]$$

$$\Gamma = m\,h_2\,p\,a^3$$

Loading (*b*).—For a unit load, p, uniformly distributed over the entire span of the beam:

$$\alpha = m\,(c + h_1)\,\frac{pl^3}{12}$$

$$\Gamma = m\,h_2\,\frac{pl^3}{12}$$

Loading (*c*).—For a loading due to the cantilevers of the roof:

$$\alpha = -\mu_1\left[2\,n\,h_1\left(c + \frac{h_1}{2}\right) + m\,l\,(c + h_1)\right]$$

$$\Gamma = -\mu_2\,m\,h_2\,l$$

in which, μ_2 is the moment of these cantilevers over their supports.

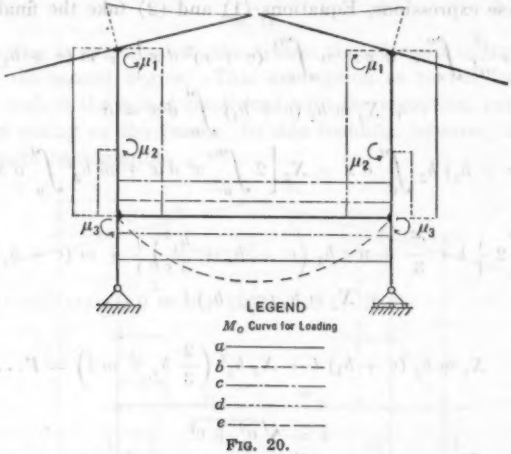


FIG. 20.

Loading (*d*).—For a loading due to the cantilevers or brackets above the elevation of the beam (for the inner brackets or cantilevers):

$$\alpha = \mu_2\left[2\,n\,f\left(c + d + \frac{f}{2}\right) + m\,l\,(c + h_1)\right]$$

$$\Gamma = \mu_2\,m\,h_2\,l$$

in which, μ_2 is the moment of the cantilevers over their supports. For the outer brackets the sign of μ_2 in the last two equations must be reversed.

Loading (*e*).—For a loading due to the brackets at the elevation of the beam:

$$\alpha = \mu_3\,m\,(c + h_1)\,l$$

$$\Gamma = \mu_3\,m\,h_2\,l$$

in which, μ_3 is the moment of these brackets at their supports.

If the influence of an unsymmetrical loading is to be considered, the rigid frame is statically indeterminate to the third degree. For the third unknown —the vertical component of the roof hinge—it is necessary that a third equation similar to Equations (1) and (2) be used.

For those who do not favor statically indeterminate structures, the frame represented in Fig. 18 can be made statically determinate by inserting a hinge in the top of the roof as is shown in Fig. 19. The result is a three-hinged frame, the analysis of which is well known.

LAURENCE J. WALLER,* M. AM. SOC. C. E. (by letter).—The author has written a concise and yet complete description of a building which was designed primarily to achieve structural economy. The design of this structure is a refreshing change from the current procedure in architectural engineering. As a general rule the architect first develops a building rather thoroughly in accordance with his ideas of architecture. The engineer is then called in and invited to support the building in such spaces as the architect has not seen fit to utilize in his treatment. The resultant structure is, of course, a compromise between engineering design and architectural concession. In this instance from very necessity the building was handled, from its inception, as an engineering problem. The architectural treatment was subordinated to the engineering requirements, and, indeed, it consists of little more than a refinement and decoration of the structure.

In a larger sense, this structure has added nothing new to architectural engineering. There are no features inherently novel in the design. It is worthy of consideration, however, in that its conceptions are fundamentally sound. Two main considerations govern the design: (1) Selection of the materials best adapted to the specific requirements; and (2) refinement of engineering design to secure maximum efficiency from the materials chosen.

In a balcony of this magnitude steel framing is decidedly indicated. The large cantilever length would definitely predicate such construction even if considerations of head-room and exit clearance did not demand it. In connection with the roof, the chief desideratum was to erect a structure which would be light and which would not exert a large bending moment on the long columns that support it. Steel trusses on rollers met the requirements very satisfactorily. Throughout the remainder of the structure, the requirements were not so severe, and reinforced concrete was chosen as being adequate for the construction involved and considerably cheaper than steel. As indicative of the economy of construction attained, attention is called to several features which may be of service in similar structures.

In the wall footings, the height of the wall below the first floor was used as a moment arm in the continuous wall footings. This procedure is in accordance with the probable stress distribution and is highly efficient.

The rigid frame construction in the Banquet Hall is the most economical type of structure and has the additional merit of producing a very graceful architectural effect. Unfortunately, statically indeterminate structures are

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comparatively little used in the United States. There is a mistaken feeling that this type of construction involves laborious and protracted engineering calculations. In some instances this, of course, is true. Literature of the rigid frame has been developed to such an extent, however, that, in general, a rigid frame design is as simple as a statically determinate problem. More serious consideration should be given to this form of construction since it is almost invariably highly economical and pleasing in appearance.

The cantilever purlin system is a construction much in vogue in Europe. There seems to be no good reason why its use in this country should not become more general. As against a bending moment of $\frac{WL}{8}$, the cantilever system can be arranged to secure a moment of $\frac{WL}{10.45}$ in the end spans and $\frac{WL}{16}$ in all interior spans. This is a saving not to be regarded lightly. The system is applicable to floor construction and has been used by the writer in that connection with notable economy. It is particularly applicable to long-span floor construction, where its use not only achieves economy, but also decreases the depth of a floor system to a minimum not otherwise attainable.

R. McC. Beanfield,* Assoc. M. Am. Soc. C. E. (by letter).—Considering the scope and valuation of the building industry comparatively little descriptive and practical data, relative to the actual design and construction of structural frames, particularly those involving statically indeterminate problems, has been published. Many theories have been advanced and discussions written about the design of building structures, but practical descriptive matter of value, covering the actual design, construction, and tests of such structures, is woefully lacking.

With this thought in mind the writer submitted his paper with the hope that, in the solution of his problems (some of which were rather unusual), he would contribute some practical data of value to those interested in structural design and economies of building construction.

Mr. Waller calls attention to the fact that the building was treated, from its inception, as an engineering problem to which the architectural treatment was more or less subordinated. By the very nature of the structure, with its unusual long-span elements, it consisted, to a large extent, of purely engineering problems. On the other hand, the esthetic treatment was not without its difficulties, and these were solved efficiently and economically by the architect. The desired efficiency was produced by 100% co-operation between engineer and architect, and resulted in many economies in the actual construction of the building.

Is it not really a fact that many modern buildings express false conceptions of architectural beauty—particularly of strength and stability—by the illogical use of decorated veneers, arches, and beams that do not seem to have substantial supports, and by other deceptive motifs belying strength and supporting beauty?

In the "Shrine Civic Auditorium" the architect expressed the large structural units which, for the most part, were exposed, as elements of architectural

* Cons. Engr., Los Angeles, Calif.

beauty. An arch is always more pleasing than a straight soffit beam. Why not use more concrete arches as real supports, thereby obtaining 100% use of the concrete, whereas only about 38% is actually utilized in a reinforced concrete beam? Why resort to the use of plaster when the natural surface of the concrete can be treated, or decorated, with excellent results and more economically, as was done in many of the structural elements of this building?

Special attention is directed to Mr. Waller's statements, namely,

"* * * (1) Selection of the materials best adapted to the specific requirements; and (2) refinement of engineering design to secure maximum efficiency from the materials chosen."

It is an exception rather than a rule, that the structural engineer is not permitted to use his judgment in the selection of the most efficient materials for a specific purpose. The arbitrary policy of dictating that this or that material must be used throughout the structure tends toward considerable waste of money.

Without the use of the elastic theory it would have been practically impossible to design most of the large structural units with any degree of safety or economy, because many of the structural elements were statically indeterminate in varying degrees. Reinforced concrete, owing to its monolithic characteristics and the fixed conditions and continuity at the joints, involves more or less statically indeterminate problems, which should be solved by the application of proper methods based on the elastic theory. It will avail nothing to ignore the stern fact that arbitrary and illogical rules, as adopted in most municipal building codes, will serve as a panacea for the design of such structures. There is ample information and precedent now available to those who will interest themselves in modern statics.

Mr. Waller emphasizes the methods of designing continuous wall footings wherein considerable economy is obtained by locating the tension steel high up in the wall above the footing proper. Mention is also made of the economy involved in the cantilevered purlin system, which can likewise be applied to floor systems.

Mr. Noetzli's discussion has supplemented the paper with several interesting features of the structural design, particularly the proscenium arch. Enlarging on his suggestions relative to extensometer readings, the writer strongly advocates the measurements of deformations in large and important structural elements of a building, especially of statically indeterminate reinforced concrete frames. For example, it would be enlightening to know the magnitude and effect of secondary stresses in long-span reinforced concrete trusses, several of which have been constructed within the past five years. The measurement of stresses in rigid frames, verifying theoretical assumptions, would do much to create increased confidence in, and the more extensive use of, such structures.

Relative to extensometer readings Mr. Belzner states:

"When the pressure was always placed directly over the contact points and firmly held against the member to be measured, there could not possibly be any spring of the various parts in the strain-gauge, causing variations in the readings."

The personal equation of the operator has much to do with the accuracy of strain-gauge readings. Applying equal, vertical pressures on two points 8 in. apart, sometimes in difficult positions, by one man, is easier said than done. Unless the pressures are exactly vertical over the contact points a variation in the deformation read is certain. For this reason an average of three measurements was taken.

To read deformations accurately by extensometers requires considerable technique and varies for different types of strain-gauges. More published data on the technique of strain-gauge measurements would be of considerable assistance to the profession.

Mr. Floris submits a commendable discussion relative to the solution of the rigid frames supporting the Banquet Hall roof. His suggestions relative to the use of hinges and the avoidance of fixity by brackets at the column joints, at the first and second-story girders, in order to reduce to a minimum the statical indeterminate conditions, are noteworthy and lead to a definite determination of all stresses in the frame. On the other hand, the value of the rigidity of the structure, as a whole, must not be overlooked for conditions that are produced by seismic disturbances. With slight variations in the seismic oscillations a series of loosely jointed beams and columns would act as battering rams, resulting in much probable damage to the entire structure.

The writer's analysis of the stresses in these frames assumed the main columns to be hinged at the second-floor level, with hinges at the outer supports of the roof girders, thus obtaining a frame statically indeterminate in one degree—that of the horizontal reactions at the column hinges. Other parts of the frame, below the column hinges, were designed on the basis of relative rigidities.

The solution of the three-hinged frame, as suggested by Mr. Floris, is quite novel and instructive. The method of analysis is given in detail and contains general formulas for the most practical loading conditions.

That the writer has been repaid for his efforts is reflected in the assistance he has been able to render others desiring data involved in the solution of some of the problems presented in his paper.

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Paper No. 1701

ADVANCES IN WATERWAYS ENGINEERING DURING A HALF CENTURY*

By W. M. Black,† M. Am. Soc. C. E.

With Discussion by Messrs. C. L. Hall, De Witt D. Barlow, and W. M. Black.

Synopsis

A review of the progress of waterways engineering must include statements of advances made both in the science and in the art of engineering. Since an engineering problem is solved best when the desired results are obtained in the least time and at the least cost, progress in engineering is measured by the improvement of methods and the reduction of costs. The art of engineering is very old. It is questionable whether more noteworthy examples will ever be produced than those exemplified in the pyramids and temples of Egypt, the irrigation works known to ancient peoples, the aqueducts of Rome, or the walls and canals of China.

Modern engineering can boast of great triumphs, but the advances made have been mainly in plant and methods causing reductions of time and costs. In this paper, the writer will not be able to treat this subject from the world-wide viewpoint which it deserves. He will simply try to show some of the progress made in waterways engineering in the United States.

Science of Waterways Engineering

Researches into the laws governing the flow of water in streams date back through the centuries, but it is hardly too much to claim for Humphreys and Abbot that present knowledge of stream flow is largely based on their pioneer work in the Mississippi, carried on during the middle of the Nineteenth

* Presented at the meeting of the Waterways Division, Philadelphia, Pa., October 8, 1926, and published in October, 1927, *Proceedings*.

† Maj.-Gen., U. S. A. (*Retired*); Engr. (Black, McKenney & Stewart), Washington, D. C.

Century. Since then, engineers of both Europe and America have continued the research until, to-day, formulas are available by means of which the discharges of streams can be determined with a fair degree of accuracy. The relation between rainfall and stream discharge is yet under investigation. Many data have been collected and tentative formulas proposed. The complexities which this subject presents are well shown in papers published by the Society.

The question of flood control has also received wide consideration by American engineers and has been the subject of papers and discussions before the Society. The Assouan Dam, in Egypt, and the work of the Miami Conservancy Commission, in Ohio, are notable examples of works constructed to this end during the past fifty years. The relation of deforestation to river floods has been widely discussed, notably by Chittenden,* Burr,† and Townsend.‡

The action of side channel spillways has been well discussed§ by Julian Hinds, M. Am. Soc. C. E. The efficacy of such spillways in controlling flood heights is under trial in the Sacramento River, California, and is being studied for the Mississippi River by a Board of Engineers.

The effects of a regulated flow from storage reservoirs in maintaining navigable depths in the Mississippi River above Lake Pepin, and in decreasing flood heights, have been under observation and record since 1880, when the reservoir project was authorized, affording a material increase in exact data on this important branch of waterways engineering.

The volumes, movement, and heights of floods in the Mississippi River have received prolonged study by the members of the Mississippi River Commission and their assistants, and to-day it is possible to predict, for points on the lower river, the height which will be attained by a flood wave passing St. Louis, Mo., or Cairo, Ill. The phenomenon of the increase of the velocity of flood wave propagation over that of flood river current, slight in shoal water, but great in deep reaches, has been studied and announced. Another important addition to waterways engineering knowledge, is the better understanding of the courses followed by the filaments of water of a river in their progress through a bend, and the better understanding of the processes of bank erosion and bar formation obtained therefrom. The important functions of bends in a river which flows through an alluvial plain are also now better appreciated. "Boils" in the deep water of bends and the cause of the formation of the "mud lumps" near the mouth of a river like the Mississippi, are yet under study. The general principles of waterways engineering as now generally accepted, have been ably outlined by Townsend.‖ Progress has been made in the study of tidal action in rivers and estuaries and in canals which join two bodies of water in which tidal action exists. Parsons' study of the tides and their action

* "Forests and Reservoirs in Their Relation to Stream Flow, with Particular Reference to Navigable Rivers," by H. M. Chittenden, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXII (1909), p. 245.

† H. R. Doc. 9, 62d Cong., 1st Session.

‡ *The Military Engineer*, Vol. XIV, 1924, p. 242.

§ *Transactions*, Am. Soc. C. E., Vol. 89 (1926), p. 881.

‖ "The Hydraulic Principles Governing River and Harbor Construction," C. McD. Townsend, M. Am. Soc. C. E., The MacMillan Co., 1922.

in the Cape Cod Canal,* Marmer's studies of the tides and their action in New York Harbor,† and the report of Zeskind and Le Lacheur on those of Delaware Bay and River,‡ give valuable data. The best study of this subject known to the writer is that of L. Bonnet, Engineer and Director of Ponts et Chaussées, Belgium, on tidal action in the Scheldt River,§ in which the magnitudes of the action of the various tidal phenomena in any reach of the river are shown to be functions of the tidal energy entering that reach, and formulas are given by which, for any proposed change in the channel capacity of the reach, the corresponding change in the tidal phenomena can be predicted within reasonable limits of accuracy. This subject is now being studied on the Chesapeake and Delaware Canal.

The great developments of the past half century in irrigation and in hydro-electric production have led to increased knowledge of the principles governing works constructed for these purposes, but these subjects are beyond the scope of this paper.

The formation of an up-stream undercurrent of salt water at and near the mouths of the Mississippi River, when the main river current is not reversed in direction, was first investigated by Lt. Col. E. J. Dent, Corps of Engineers, U. S. A., M. Am. Soc. C. E., and described‖ by Col. C. McD. Townsend, U. S. A. (*Retired*), M. Am. Soc. C. E. The subject of bottom salt-water undercurrents was also studied in 1921-22 by Capt. (now Major) A. P. von Deesten, Corps of Engineers, U. S. A., Assoc. M. Am. Soc. C. E., in the Sabine-Neeches Waterway, Texas.¶ The action of a lock between an upper fresh-water pool and a lower pool of salt water, in pumping salt water into the upper pool, has been studied at the Charles River Lock, Boston, Mass., and the Gatun Locks of the Panama Canal. There have also been made many studies of the effects of storm wave and current action on sandy coasts.**

The causes and effects of upward water pressure on dam foundations and the measures necessary to overcome this pressure have been widely investigated. Many studies have been made of the stresses in arches and particularly in the arch of a single arch dam, under the supported pressures.

All these studies form additions to the science of waterways engineering.

Art of Waterways Engineering

The past fifty years have produced notable advances, largely due to increased knowledge of the properties of materials, to the use of electricity for power and light, to the development of steam and internal combustion engines, and to the great improvement of machines thereby made possible.

* "The Cape Cod Canal," by William Barclay Parsons, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXXII (1918), p. 1.

† "Tides and Currents in New York Harbor," H. S. Marmer, *Serial No. 285*, Dept. of Commerce, U. S. Coast and Geodetic Survey.

‡ *Serial No. 336*, Dept. of Commerce, U. S. Coast and Geodetic Survey.

§ "Contribution à l'Etude Théorique des Fleuves à Marées et Application aux Rivieres à Marées," par L. Bonnet, Ingenieur en Chef, Directeur des Ponts et Chaussées, *Annales* des Travaux Publics de Belgique, Fascicles 3, 4, 5, et 6, 1922, et 1, 2, 3, et 5, 1923.

‖ *The Military Engineer*, Vol. XVIII, (1926), p. 99.

¶ H. R. Doc. No. 12, 64th Cong. 2d Session, and *The Military Engineer*, Vol. XV, No. 84 (1923), p. 527.

** *Transactions*, Am. Soc. C. E., Vol. XXIII (1890), p. 123; Vol. XXIX (1893), p. 223; Vol. L (1903), p. 66; Vol. LXXX (1916), p. 1786; and Vol. LXXXVII (1924), p. 589.

Although concrete in one form or another has been in use for centuries, its wide application to all classes of structures is less than fifty years old, and this wide use is largely due to the employment of steel in reinforcement. As a result of this use, the properties of concrete and the action of the reinforcement have received extended study, with the development of numerous formulas for practical application. A recent advance has been in the development of alumina cements which attain their strength in a short time after setting.

Hydrographic Surveying.—Aeroplane photography has greatly facilitated the work of hydrographic surveying. By its aid, extended surveys can now be made periodically with relatively small expenditures of time and money, and changes in the course of a stream, or in a coast line, noted and their cause studied.

The use of drags or sweeps in determining navigable depths and discovering submerged obstructions has become general and the sweeps themselves have been improved in form.

The first sweep designed to cover large areas rapidly was devised by Capt. S. S. Leach, Corps of Engineers, U. S. A., and used by the U. S. Lake Survey in 1893-95.* Subsequently, in 1902, it was greatly improved by F. C. Shenehon, M. Am. Soc. C. E., then Principal Assistant Engineer of the Lake Survey.† As developed, the sweep can cover a swath 2 000 ft. wide. In 1908, the maximum day's work of one sweep covered 5 sq. miles.‡ The use of the sweep was adopted by the U. S. Coast and Geodetic Survey about 1904. In a recent survey in Alaska, the Coast and Geodetic Survey party was able to cover 65 sq. miles in one day's work.§ The construction and operation of the wire drag and sweep are described in recent publications of the U. S. Coast and Geodetic Survey.‖

A method of making continuous lines of soundings was developed by the U. S. Lake Survey in 1899-1900, by the use of a heavy weight suspended by a fine wire from a drum, the revolutions of which gave the depth. The weight was raised just clear of the bottom after each sounding. Soundings were made at intervals of 20 sec.¶ A few years afterward the late E. S. Wheeler, M. Am. Soc. C. E., invented an instrument called a bathometer by which a continuous profile of the bottom can be delineated on paper.**

The most important improvement of sounding apparatus is one developed quite lately by the Engineer Department of the U. S. Navy, called the "Sonic depth-finder." With this instrument deep-water depths can be determined at short intervals without stopping the course of a vessel. This instrument has proved of special value in work connected with the placing and repair of submarine cables in ocean depths. With this instrument the

* Rept., U. S. Chf. of Engrs., 1896, p. 4063.

† *Loc. cit.*, 1903, p. 2763.

‡ *Loc. cit.*, 1909, p. 2496.

§ "The Evolution of the Nautical Chart," by E. Lester Jones, M. Am. Soc. C. E., *The Military Engineer*, Vol. XVI, 1924, p. 219.

‖ "Construction and Operation of Wire Drag and Sweep," by J. H. Hawley, *Special Publication No. 118*, U. S. Coast and Geodetic Survey, November, 1925.

¶ Rept., U. S. Chf. of Engrs., 1902, p. 2781.

** *Loc. cit.*, 1908, p. 2528.

Hydrographic Office, U. S. Navy, is making bathometric charts of selected ocean areas. In "Bathometric Chart No. 5194, San Francisco to Pt. Descanso", the ocean bed was covered between the 100 and 2 000-fathom contours, with soundings in parallel courses, 10 or 5 miles apart. In 38 days of field work 34 000 sq. miles were contoured by 5 000 soundings made at a ship speed of 12 knots. In working up the data collected by the field parties, a new instrument called the "Pantograver" has proved of value. With attachments eliminating distortion of cloth or paper, it is hoped eventually to be able to engrave plates from photographs; such an attachment is now in use, but it concerns one direction only.*

Many improvements in instruments and methods have been made by the U. S. Coast and Geodetic Survey. Recently, a portable automatic tide gauge has been developed by G. T. Rude, Commander, U. S. Coast and Geodetic Survey, M. Am. Soc. C. E.,† which can be easily carried and installed in the field. A current direction indicator has also been devised with which current directions can be determined simultaneously at three different depths on the same vertical.†

The U. S. Coast and Geodetic Survey has developed a method whereby the position of a survey ship can be accurately determined when it is beyond the range of visibility of shore objects and when closer to shore during low visibility on account of fog and haze. The method also permits surveying during the hours of darkness with the result that offshore hydrography has been materially speeded up by permitting the ship to work continuously throughout the day, and even on those days when formerly field work was impossible. This method and the apparatus have been fully described.‡

The Bureau has conducted extensive tests on various types of echo sounding apparatus, including the Navy sonic depth finder and an apparatus invented by the Submarine Signal Corporation known as the Fathometer. The Fathometer was designed primarily as a navigational sounding machine, and, in its original form, did not possess quite the accuracy required for surveying. However, this defect has been eliminated by the addition of a suitable governor on the timing apparatus so that now results can be obtained with the Fathometer quite as accurately as by any echo sounding apparatus and with almost the precision of direct wire measurements. The Bureau has two ships equipped with this apparatus and expects to so equip all surveying ships. The particular merit in this type of echo sounding apparatus lies in its ability to measure moderate and shallow depths. The Fathometer to date gives very accurate results down to about 15 fathoms, and there is reason to believe that with certain definite improvements the range may be extended to perhaps 4 or 5 fathoms.

Two sounding machines of the reel and wire type have been developed by the Bureau, one for use in great depths and the other for depths of 400 to

* *The Military Engineer*, Vol. XVII (1925), p. 39.

† "Tide and Current Investigations of the Coast and Geodetic Survey", by E. Lester Jones, M. Am. Soc. C. E., U. S. Coast and Geodetic Survey (*Serial 351*), 1916; also, "Tides and Their Engineering Aspects," by G. T. Rude, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 92 (1928), p. 606.

‡ "Radio Acoustic Method of Position Finding in Hydrographic Surveys," by Hech, Eckhardt, and Keiser, *Serial* 227, Dept. of Commerce; U. S. Coast and Geodetic Survey, *Special Publication No. 107.*

500 fathoms. These machines consist of reels driven by electric motors and controlled in such a way that the reels may be stopped instantly and started at any desired speed. While no new principles are involved in this apparatus, it is, nevertheless, a great improvement over the older type, due to its simplicity, sensitiveness, and flexibility, all of which result in greater speed of operation and greatly reduce the danger to the submerged apparatus, detaching rods, water specimen bottles, and thermometers. It has been found that inexperienced personnel can operate these new machines as efficiently as trained operators could handle the older types. As an illustration of this point, it may be mentioned that one survey ship with a new and untrained crew carried a line of soundings from New York to San Diego and, in addition, explored the deep north of Porto Rico without once parting the wire or losing any equipment. That, it is believed, established a new record for oceanographic work.

Pressure tubes for measuring depths as great as 100 fathoms with considerably greater precision than the commercial glass tubes have been developed in this Bureau and are now used extensively on surveys on which heretofore such apparatus could not be tolerated because of its unreliability. These tubes record depths with a probable error not in excess of 5% and are now used extensively for depths greater than 15 fathoms.

Dredging.—Dredges operated by steam power have been used in river regulation for more than a century, but during the past fifty years their efficiency has been vastly increased. In addition to the old type clam-shell, dipper, and ladder dredges (all of which have been greatly improved), hydraulic pump dredges have come into general use.

These dredges are of two general classes: Hopper dredges and pipe-line dredges. The hopper dredge, while under way, excavates material by means of a drag attached to the end of a suction pipe of a pump and discharges into hoppers in the ship. It was invented by a Mr. Lebby, at Charleston, S. C., in 1855, where the first of its type was used in 1857. After the Civil War a similar dredge was built in 1871, by the late Brig.-Gen. William Ludlow, U. S. A., M. Am. Soc. C. E., (then Captain, Corps of Engineers, U. S. A.), for use on the South Atlantic Coast. It is the only type of dredge which can be operated efficiently in a seaway and is now in general use throughout the world for deepening the entrances to harbors and rivers, having been markedly developed in efficiency and power. By its aid, the problem of securing and maintaining entrance channels protected by jetties has been greatly simplified. In the latest dredges of this type, the propelling, steering, and pumping machinery is actuated by electricity generated from power supplied by internal combustion engines.

These electric dredges, due to their ease of handling and close control, are capable of being operated easily in a very limited space, such as in narrow channels, or within a very short distance of piers, which would be both difficult and more dangerous with steam dredges. While cost records for a complete year of operation are not available, it is estimated that operating expenses

will be 20% less than those of the same type of steam dredge. Maintenance expenses thus far can be only estimated.

These dredges have been used largely on experimental work, with a view to determining the type of dredging for which they were best fitted, resulting naturally in an increased cost of operation. On certain classes of dredging the cost has been as low as 3 cents per yd., with approximately 400 000 cu. yd. removed monthly.

Pipe-line hydraulic dredges were first used on the Pacific Coast. These swing in a position which is slowly moved forward, excavate material by means of a revolving cutter at the bow, just in advance of the end of a suction pipe, and discharge through a pipe line supported on floats.

Fig. 1 is a view of the U. S. Pipe-Line Dredge *Wahalak*, showing an old type cutter for soft materials. Fig. 2 shows a new type cutter for soft materials on the U. S. Pipe-Line Dredge *C. B. Harris*.

This type has also received a great development. While the general radius of delivery through the pipe line had averaged about 2 500 ft., with an elevation not exceeding 10 ft., recently, in the Chesapeake and Delaware Canal, a dredge of this type delivered material at a distance of more than 500 ft. and at an elevation of more than 90 ft. In the Panama Canal, another pipe-line dredge, aided by a relay pump on a barge, delivered through a pipe line, 1 500 ft. long, to an elevation of 250 ft., or through a pipe line, 15 840 ft. long, to an elevation of not to exceed 20 ft. This work was done at an average cost of 28 cents per cu. yd.*

It is claimed that a late type of this class of dredge, working in Florida with a 16-in. pump, excavated sand and mud, and delivered at a distance of 3 000 ft. at a cost of 7 cents per cu. yd., and when equipped with a special cutter head excavated and removed the limestone rock found there to a depth of 22 ft. at a cost of 40 cents per cu. yd. Fig. 3 shows the rock cutter on the U. S. Pipe-Line Dredge *Barnard*.

On the Mississippi River, a special class of this type of dredge has been developed for the removal of the bars of fine material found there. In this type, no revolving cutter is used, the material being stirred up by water jets in advance of the suction pipe.

Most of these Mississippi type dredges are self-propelled and resemble in outward appearance a stern-wheel, shallow-draft Mississippi River steamboat. On the hull are mounted power units; a centrifugal dredging pump; a wide structural ladder hinged in a well recess at the bow of the dredge and carrying at its lower end a dustpan-shaped suction head 20 ft. or more wide, which is provided with water-jet nozzles to loosen the material; a hauling winch; hoisting winches; spuds; the necessary auxiliaries and controls for operation; and suction and discharge piping and floating pipe line.

In operation, the dredge steams to a point above the bar, drops two anchors one on each side of the channel to be cut, then drops several hundred feet down stream on the anchor cables, to the lower edge of the bar. The dredge pump, and the pump furnishing water for the jet agitator, are

* J. G. Claybourne, *The Military Engineer*, Vol. XVII (1925), p. 910.

started, the dustpan suction head is lowered to the bottom, and the dredge is hauled up stream by the hauling winches, with the suction head against the bar. The discharge is from the side of the dredge and through a short floating pipe line, 500 to 1 000 ft. long, mounted on pontoons. The pipe line is held in a line abreast of the dredge by the dynamic action of the stream on a baffle-plate located at the end of the line. The dredge material is deposited in the stream, but along the edge of the channel, where it forms a dike which assists in deepening the channel by the scour produced by its action in regulating the direction of the stream.

When the upper edge of the bar is reached the dredge drops back again and makes similar cuts alongside of and parallel to the first cut. Thus, a channel is cut through the bar, and this channel is later widened and deepened by the action of the current through it.

The cost of dredging varies with the relative number of hours of work and of idle time, with efficiency of the plant, the depth of dredging, the character of the materials to be excavated, and the location at which spoil is deposited. For each type of dredge there is a class of excavation for which that type is best fitted, and no judgment can be formed of the relative efficiency of the types without a full knowledge of the conditions found in the work.

In this connection the record of the recent performances of hydraulic dredges used on the waterways engineering works of the United States under the charge of the Engineer Department, U. S. Army, is of value. This record, Tables 1, 2, and 3, includes dredges which worked for the entire year.

Rock Excavation.—As stated previously, soft rock in place can now be excavated by the hydraulic dredge when equipped with a special toothed cutter. Rock in thin strata can be broken by the chisel dredge, first used in the Mississippi River in 1855, but much improved since. The Lobnitz rock cutter is only a development of the old type.

Drill boats, generally equipped with piston and well drills, for drilling and blasting rock under water, also have been markedly improved. Hollow pipe drills have come into use since devised by Col. George McC. Derby, Corps of Engineers, U. S. A. (*Retired*), M. Am. Soc. C. E. (then Lieutenant, Corps of Engineers, U. S. A.), in 1883, in the excavation of Flood Rock, Hell Gate. There, too, the simultaneous explosion of 44 175 charges of rack-a-rock placed in the drill holes in the walls of the galleries cut in Flood Rock, was effected by the method of sympathetic explosions, by the explosion of charges of dynamite laid along the center lines of the galleries. These gallery charges were fitted with fuses which were connected with a firing battery.*

The improvements in drilling machinery, in electric firing batteries and exploders, and in explosives, have greatly facilitated and cheapened this class of work.

River Regulation.—The advances made in the methods of regulation works can be appreciated from a comparison of the methods used by the late Brig.-Gen. O. H. Ernst, U. S. A., M. Am. Soc. C. E. (then Captain, Corps of Engineers, U. S. A.),† in 1879, at Horsetail Bar, Mississippi River, near

* Rept., U. S. Chf. of Engrs., 1886, p. 682.
† *Loc. cit.*, 1880, Pt. 2, p. 1362.

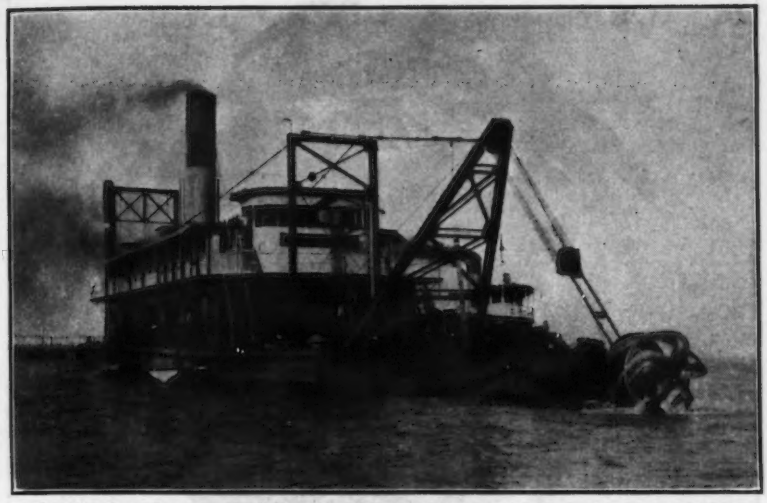
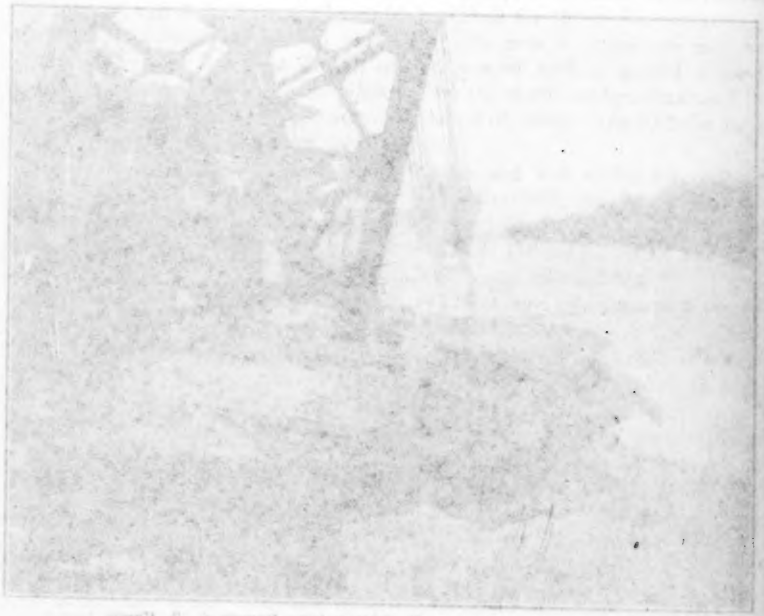


FIG. 1.—BOW VIEW OF U. S. DREDGE *Wahalak*, SHOWING OLD TYPE CUTTER FOR SOFT MATERIALS.

FIG. 2.—NEW TYPE CUTTER, U. S. PIPE-LINE DREDGE *C. B. Harris*.



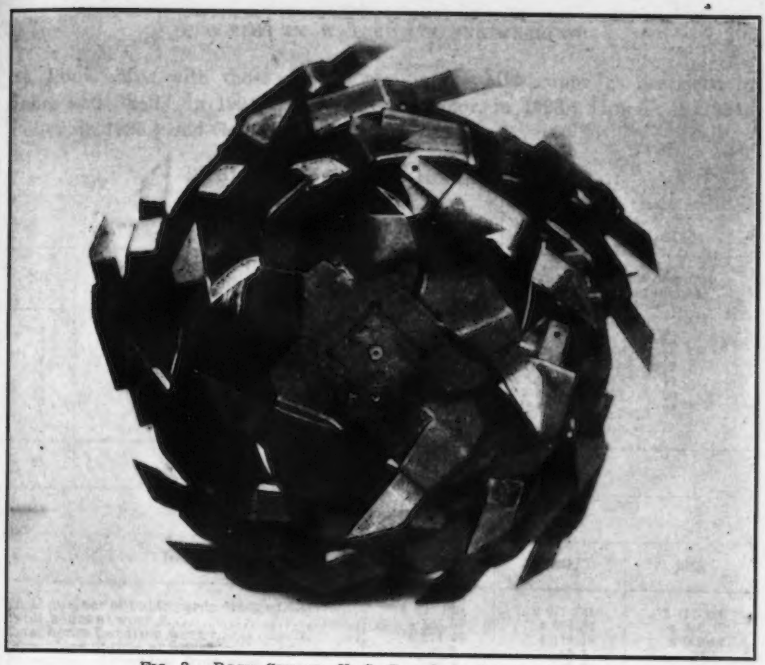
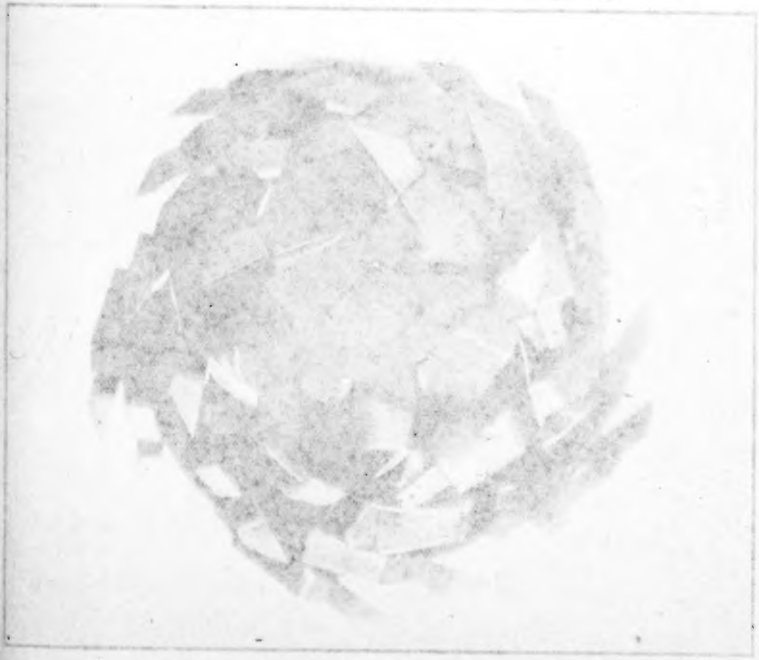


FIG. 3.—ROCK CUTTER, U. S. PIPE-LINE DREDGE *Barnard*.

FIG. 4.—TYPE OF BRUSH AND POLE MATTRESS.



St. Louis, Mo., with those now in use on the Mississippi, as described by Jones and Skelly, in 1921,* Todd and O'Connor, in 1923,† Howell, in 1924,‡ Potter, in 1925,§ and Gotwals, in 1926.‖

TABLE 1.—SEA-GOING HOPPER DREDGES.

1923.		1924.		1925.	
No.	Size of pump, in inches.	No.	Size of pump, in inches.	No.	Size of pump, in inches.
6	15	5	15	5	15
2	16	2	16	2	16
2	17	1	17	1	17
4	19	4	19	4	19
4	20	5	20	5	20
2	26	2	26	3	26
1	30	1	30	..	..
21	..	20	..	20	..

AVERAGE PERFORMANCES.

Item.	1923.	1924.	1925.
Total number of cubic yards dredged	1 727 230	2 071 741	2 197 796
Total hours at work.*	4 185.52	4 664.84	4 756.13
Total hours lost from work†	4 574.48	4 119.18	4 003.87
Percentage of time at work*	47.7	53.1	54.3
Percentage of time lost from work	52.3	46.9	45.7
Total field operating cost	$130 451	$148 815	$140 172
Total cost of repairs	$ 31 828	$ 52 815	$ 40 508
Gross operating and repair cost	$162 279	$201 630	$180 680
Total cost, in cents per cubic yard	9.25	9.73	8.22

* Time at work includes: Pumping; turning in cut; to and from dump and dumping; taking on fuel and supplies; to and from berth; and miscellaneous.

† Time lost from work includes all other time.

Prior to 1878, the guiding and cross-dikes were built solidly of stone and brush, placed on the river bottom. When this bottom was soft and erodible, these dikes were quickly undermined and destroyed. Under the direction of Captain Ernst, use was made of brush mattresses (Fig. 4) for holding the bottom, and of permeable screens of brush and of Brownlow weeds for reducing current velocities and inducing fill. Although these proved effective, they were too fragile. Brush mattresses are still used, but of improved construction. The latest advance has been in building brush mattresses of fascines, or of substituting for the brush mattress, an articulated, reinforced concrete mat. Fig. 5 shows a type of woven mattress and Fig. 6 is a view of concrete bank revetment at Gasconade, Mo. The permeable dikes have been made stronger and more permanent in form by the use of pile bents firmly connected. Quite recently, reinforced concrete piles and caps have been used with good results in cost. Fig. 7 shows the results of the use of a retard (permeable dike)

* *The Military Engineer*, Vol. XIII, 1921, p. 197.
† *Loc. cit.*, Vol. XV, 1923, p. 215.
‡ *Loc. cit.*, Vol. XVI, 1924, p. 283.
§ *Engineering News-Record*, Vol. 94, 1925, p. 508.
‖ *The Military Engineer*, Vol. XVIII, 1926, p. 236.

on the Missouri River. The main channel was along the bank at the time of construction.

The system of "retards" made of trees, now extensively used for bank protection, is only a larger "Brownlow weed" firmly anchored by concrete piles sunk deeply into the river bottom by the water-jet process. The channel for the jet is cast in the center of the pile, with exits for the water distributed at intervals along the pile length.

TABLE 2.—HYDRAULIC PIPE-LINE DREDGES. CUTTER TYPE.

1923.		1924.		1925.	
No.	Size of pump, in inches.	No.	Size of pump, in inches.	No.	Size of pump, in inches.
2	10	2	10	2	10
6	12	7	12	6	12
8	15	8	15	8	15
4	16	3	16	3	16
3	18	2	18	3	18
17	20	17	20	16	20
2	24	4	24	5	24
42	..	43	..	43	..

AVERAGE PERFORMANCES.

Item.	1923.	1924.	1925.
Total number of cubic yards dredged	1 471 444	1 491 481	1 687 684
Total hours at work*	3 819.48	3 859.50	3 804.83
Total hours lost from work†	4 940.52	4 790.10	4 942.88
Percentage of time at work*	43.6	44.6	43.5
Percentage of time lost from work†	56.4	55.4	56.5
Total field operating cost	$ 94 687	$ 95 276	$ 94 661
Total cost of repairs	$ 27 710	$ 30 919	$ 30 877
Gross operating and repair cost	$122 397	$126 195	$125 538
Total cost in cents per cubic yard	8.32	8.48	7.44

RELATIVE COST PER CUBIC YARD FOR DIFFERENT SIZES OF PUMP.

	Cost per Cubic Yard, in Cents.									
Size, in inches.	8.	10.	12.	15.	16.	17.	18.	20.	22.‡	24.
1923	0.226	0.195	0.139	0.209	0.164	0.070	0.092	0.067	0.376	0.060
1924	0.214	0.190	0.173	0.151	0.183	0.052	0.128	0.063	0.681	0.066
1925	0.219	0.190	0.160	0.108	0.128	0.116	0.122	0.065	0.150	0.045

* Time at work includes: Pumping; shifting and waiting on pipe line or scows; shifting dredge; passing vessels; going to and from berth; and miscellaneous.

† Time lost includes all other time.

‡ The higher costs for the 22-in. pump dredge were due to its use in excavating sand and coral rock.

The method of sinking piles by the aid of a water jet* was first used in Matagorda Bay, Texas, at the suggestion of the late Gen. George B. McClellan,

* "The Water Jet as an Aid to Engineering Construction," by the late L. G. Schermerhorn, M. Am. Soc. C. E., U. S. Asst. Engr., Engr. Dept., U. S. Army, 1881.

U. S. A. (then Lieutenant, Corps of Engineers, U. S. A.). It is said that he conceived the idea after watching sand crabs bury themselves. The water jet was used occasionally, thereafter, as an aid in pile-driving and in sinking caisson cylinders, but it was not until 1871 that its true function of relieving the pile from skin friction was established by Captain Brown, Corps of Engineers, U. S. A., in the operation of sinking screw-piles for the pier at Lewes, Del., when he found that the jet had to be applied above, and not below, the screw disk. The patented pile of the "retard" process takes full advantage of this principle. The use of the water jet in pile-driving is now almost universal. Reinforced concrete piles are also of recent origin.

TABLE 3.—PERFORMANCE OF HYDRAULIC PIPE-LINE DREDGES WORKING UNDER CONTRACT.

1923.	
Total yardage	13 793 319
Total of contract prices	$2 824 197
Average price per yard, in cents	20.99
30 contracts	
9 districts	
No rock included	
1924.	
Total yardage	25 016 843
Total of contract prices	$5 634 324
Average price per yard, in cents	21.3
29 contracts	
8 districts	
No rock included	
1925.	
Total yardage	14 578 148
Total of contract prices	$2 970 327
Average price per yard, in cents	19.9
29 contracts	
10 districts	
No rock included	

Levees.—In place of the old dump-wagon and horse-drawn scraper process, levees are now constructed more safely, more rapidly, and at less cost by the use of drag-line scrapers and of movable cableways from which scoop buckets are operated.* As a result of experience, the Mississippi River Commission has adopted a standard cross-section for levees, which is proving satisfactory.

Jetties.—Little change has been made in the methods of the construction of jetties at river and harbor entrances, except that brush mattresses are now rarely used, and then only as a foundation on an exceptionally soft bottom. For jetty construction, a trestle built along the jetty axis and carrying a railroad track is now generally preferred.

Canalization.—In canalization work various types of movable dams have been tried out and improvements made. By an improvement in the hurter

* *The Military Engineer*, Vol. XVI, 1924, p. 288.

(the cast-iron shoe against which the end of the prop rests when the wicket is up) of the Chanoine wicket dam, the necessity for the use of the troublesome tripping-bar can be avoided if so desired. A new type of wicket, invented by Mr. Guy B. Bebout about 1913, was placed in service at Dam No. 13, Ohio River, in 1915. The advantages of this wicket are:

1.—It can be raised at any open-river stage to the top of wickets without serious loss of time.

2.—It can be raised against any difference in pool levels, sufficient power being applied at the chains.

3.—It can be lowered under all conditions of river stages and pool levels.

4.—When raising the wicket, drift cannot obstruct or become entangled in the parts to such an extent as to prevent this maneuver, except in very extreme cases, and in such cases the drift should be easily removed.

5.—A boat, barge, or heavy drift striking the top of the wicket will cause the wicket to collapse without damage to the boat or dam.

Bear-trap dams have been greatly improved and dams of this type are now in use with lengths deemed impracticable fifty years ago. Fig. 8 shows Lock and Chanoine Dam No. 24, Ohio River. The fixed wire shows in the foreground, with the bear-trap, partly down, just beyond.

Perhaps the greatest advances made in this branch of engineering art have been in the details of lock construction.

Coffer-Dams.—The invention of interlocking steel sheet-piling has made practicable the construction of coffer-dams of a new and simple type, exemplified in the coffer-dams for the Black Rock Lock, Niagara River, the coffer-dam for the *Maine*, Havana Harbor, the Browns Landing Lock, Cape Fear River, North Carolina, and that for the Forty-Sixth Street Pier, New York, N. Y. In this last,* designed by Charles W. Staniford, M. Am. Soc. C. E., the pressure head of water resisted was 65 ft.

Foundations.—This form of piling has also proved of great value for cut-off walls around and below lock foundations in permeable soils and for cut-off walls in earth dams.

Locks.—In 1892 and 1893, the late W. L. Marshall, Brig.-Gen. U. S. A. (then Captain, Corps of Engineers, U. S. A.), began the construction of the concrete locks of the Illinois and Mississippi Canal. These are believed to have been the first locks constructed entirely of concrete.† To-day, the use of stone masonry for lock walls is exceptional.

Gates.—In the lock of the Davis Island Dam (now removed) single-leaf gates, 118 ft. long, which when operated rolled on a track into or out of a masonry recess in the land wall, were used for the first time (1881-82).

Culverts.—The lock of the Davis Island Dam was filled through culverts in the side-walls, fitted with valves moved by hydraulic power. It was emptied through a culvert below the lower gate recess. In the locks of the St. Mary's Falls Canal of 1881 and, later, as in the Panama Canal locks, the filling and emptying culverts have their orifices in the floors of the locks.

* *Transactions*, Am. Soc. C. E., Vol. LXXXI (1917), pp. 498 and 553.

† Rept., U. S. Chf. of Engrs., 1894, p. 2164.

FIG. 5.—TYPE OF WOVEN MATTRESS.

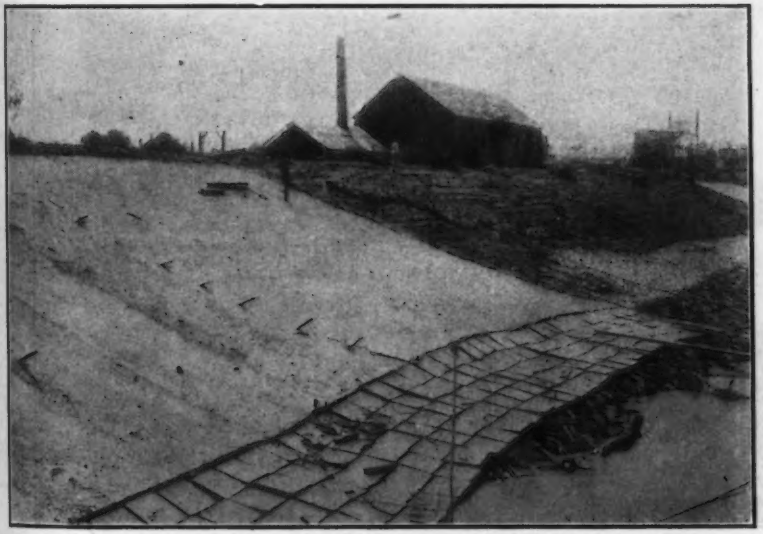


FIG. 6.—VIEW OF GASCONADE BOAT YARD, SHOWING CONCRETE BANK REVETMENT.

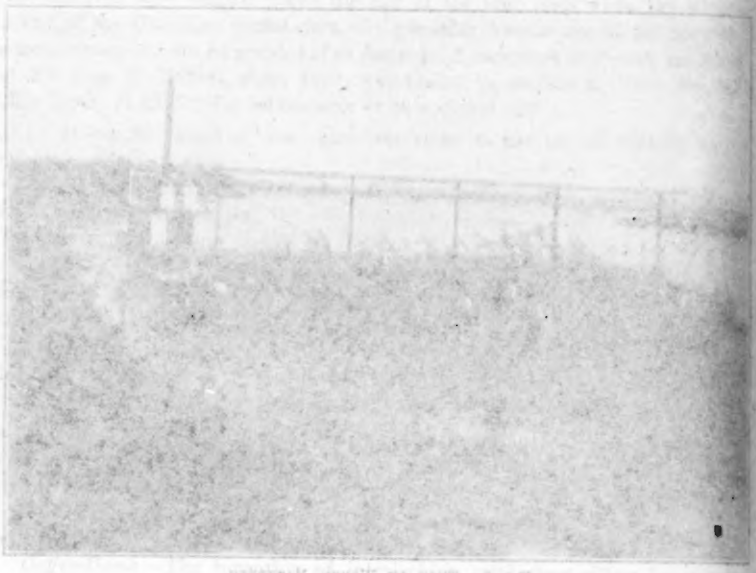


FIG. 7.—BRUSH RETARD (PERMEABLE DIKE), SHOWING FILL RESULTING FROM (LOW-WATER) WINTER SEASON.

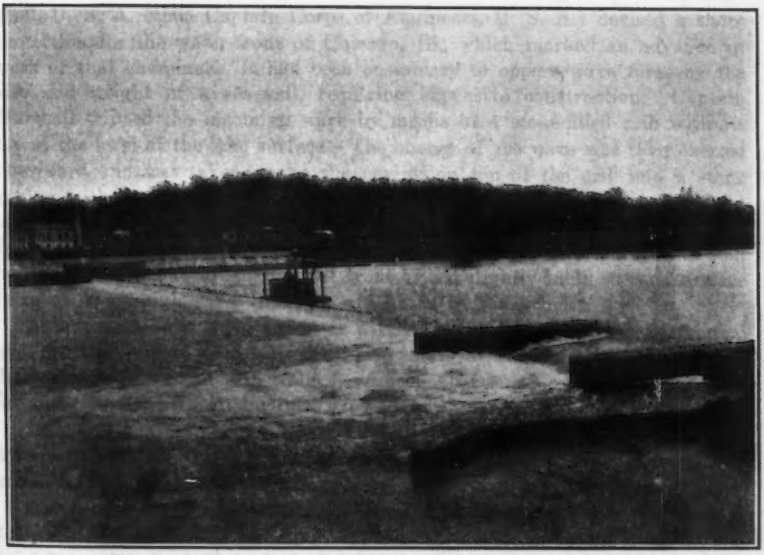


FIG. 8.—VIEW OF LOCK AND CHANOINE DAM NO. 24, OHIO RIVER.

Valves.—The well-known butterfly and cylindrical valves are yet in general use, the cylindrical type having been adopted for the great Panama Canal locks. In 1883, Mr. F. G. M. Stoney, of London, England, invented the sliding valve, known as the "Stoney gate". This type is also widely used, being at times fitted with roller bearings.

The mechanisms for opening and closing lock-gates have been greatly improved, the bull-wheel type (devised by Edward Schildhauer, M. Am. Soc. C. E., for the Panama Canal locks*) operating either chains or spars, being greatly favored for operating heavy gates.

Almost all important locks now have some form of a removable, emergency dam for closing the lock at its upper pool end in case of accident to the gates. The Panama locks are also provided with guard chains to prevent the accidental ramming of the gates.

In addition to these precautionary measures, ships passing into and through the Gatun Locks of the Panama Canal are not permitted to use their own power, but are towed by locomotives operating on tracks laid on the lock-walls.

It has also become customary in the larger locks to operate the gates and valves by power generated from the head between the upper and lower pools.

Fixed Dams.—The major advance in construction methods for fixed dams has been in the development of dams of the single and multiple-arch types. The construction of earth dams by hydraulic pumping methods should also be noted. The Gatun Dam of the Panama Canal is a notable example.

Sea Walls and Breakwaters.—About 1890, the late Brig.-Gen. W. L. Marshall, U. S. A. (then Captain Corps of Engineers, U. S. A.) devised a shore protection for the water-front of Chicago, Ill., which marked an advance in work of that character. It had been customary to oppose wave force by the size and weight of a sea-wall, requiring expensive construction. Captain Marshall tripped the incoming wave by means of a stone-filled crib with its top at the level of the lake surface. The energy of the wave was then exerted downward and was taken up by the paving on top of the crib and a stone paving to the rear laid on a gentle slope. At the crest of the slope only a light wall was needed to stop the final wash of the wave. The same principle was followed by the writer in constructing the Malecon of Havana, the sea-wall being set back about 35 ft. from the front of the slightly inclined rock surface lying at about high-water level, on which the wall was built.

In some sea-walls wave action is partly deflected by building the vertical wall with a concave face. In the breakwaters of the harbors of the Great Lakes, the old vertical crib walls have been abandoned and the breakwaters are made lower, with an inclined exposed face.

Economy has been effected and durability increased in these breakwaters by substituting for the old form of timber cribs, caissons made of reinforced concrete, which after having been placed are filled with concrete in mass.

Reinforced concrete caissons are also being used as subaqueous foundations for walls and for piers. Examples of this use are found in the dock wall at

* Paper No. 20, International Eng. Congress, San Francisco, Calif., 1915; Report, Isthmian Canal Comm., 1910, p. 53.

Petosky, Mich.*, and in the sea-wall for the China Basin, San Francisco, Calif.†

Messrs. Harrison S. Taft and O. D. Jones have written a valuable review of the use of concrete caissons,‡ and Mr. Taft's paper on "Reinforced Concrete Docks",§ presented to the Society, contains much historical data.

The progress of waterways engineering in the United States is reflected in the work accomplished. The work done has been widely spread over the entire country, as must be the case, since the care and improvement of waterways, under the Constitution, is the duty of the Nation. Appropriations for this class of work are made from the National Treasury, and the will of the people at large is that no part of the widespread territory shall be without receipt of some direct benefit from the expenditures for waterways improvements. Under this condition it has been impossible to finish individual projects for improvement as rapidly as would be beneficial were it possible to consider the needs of each improvement separately and allot funds for it irrespective of other demands. Nevertheless, the accomplishments have been great and have materially assisted the economic welfare of the people, as shown by the recorded increases of trade reported in the commercial statistics for each port and waterway published annually by the Government.‖

Space will not permit an enumeration of all work done. A few examples must suffice. Almost all the work mentioned has been done since 1865.

Almost all the harbors of the Great Lakes are at the mouths of rivers. These were obstructed by bars, largely formed by the drift of sand along the coasts under storm action (drift and wave bars). The channels across the bars were shifting, with depths of from 3 to 5 ft. The earliest improvements were to fix and deepen these channels by jetties and to deepen the rivers above the mouths by dredging. Later, a system of breakwaters was built outside the mouths, forming exterior harbors. All of the more important harbors now have a navigable depth of 20 or 21 ft. below the standard lake level. The channels connecting the Upper Lakes have been improved to the same navigable depth. For many years, the navigation between Lakes Erie and Ontario has been through the Welland Canal, lying in Canada. The Canadian Government is now engaged in enlarging the channel and locks so as to permit navigation by ships of 25-ft. draft. The total commerce of the United States ports on the Great Lakes in 1924 amounted to 203 534 234 tons,¶ valued at $3 572 521 000, in addition to a ferry traffic of 926 140 tons, valued at $125 797 000.

The general program for the rivers of the Atlantic Coast is to improve each to the extent justified by present and prospective commercial use and to connect the mouths of all between Boston, Mass., and the Rio Grande, in Texas, by an intra-coastal barge canal. The Mississippi River and its principal tributaries are being improved so as to form a system of barge naviga-

* *Engineering News-Record*, Vol. 88, 1922, p. 956.
† *Loc. cit.*, Vol. 91, 1923, p. 252.
‡ *Professional Memoirs*, Corps of Engrs., U. S. A., Vol. VII, 1915, p. 145.
§ *Transactions*, Am. Soc. C. E., Vol. LXXVIII (1915), p. 1058.
‖ Rept., U. S. Chf. of Engrs., 1925, Pt. 2.
¶ *Loc. cit.*, pp. 15 and 17.

tion from Pittsburgh, Pa., Chicago, St. Paul, Minn., and Kansas City, Mo., to the Gulf of Mexico, with such improvement of minor tributaries as may be economically justifiable. The rivers of the Pacific Coast are treated individually, no great system of river grouping being possible.

Table 4 shows some of the accomplished work. The depths given are for low water.

TABLE 4.

River.	Part improved.	Natural controlling depth, in feet.	Present controlling depth, in feet.
Hudson	Hudson to Troy, N. Y.	3.5	11.5
Delaware	Philadelphia, Pa., to sea, 101 miles	17	32.5
Delaware	Trenton, N. J., to Philadelphia, 305 miles	3	10.5
Savannah (Ga)	Savannah to ocean	10	25.6
St. John's (Fla.)	Jacksonville to ocean	5 to 7	30
Mississippi	New Orleans, La., to Gulf:		
	South Pass	9	33
	Southwest Pass	9	38
	Cairo, Ill., to New Orleans	4.5	9
	Mouth of Missouri to Cairo	3.5	8
	Minneapolis, Minn., to mouth of Missouri	2.5	5
Ohio	Pittsburgh, Pa., to Leavenworth, Ind	1 to 2	9
Alleghany		3	4 to 7
Monongahela		12	6 to 8
Great Kanawha		3	6
Kentucky	For 280 miles above mouth	Unnavigable.	6
Cumberland	Below Nashville, Tenn	0.5	4
Tennessee		1	4
Missouri	Below Kansas City, Mo	2.5	3.5
Illinois		1.66	8
Sacramento (Calif.)		4	7
Columbia (Ore.)	To Portland on Willamette River	15	28

The construction of the Panama and Cape Cod Ship Canals and the enlargement of the New York State Barge Canal are all recent works. Some of the improvements made in ocean ports are given in Table 5.

TABLE 5.

Port.	Natural controlling depth of entrance channel, in feet.	Present controlling depth, in feet
Boston, Mass	23	35
New York, N. Y	23.6	40
Baltimore, Md	17	34
Norfolk, Va., (Entrance channel)	30	38
Norfolk, Va., (Inner channel)	21	40
Charleston, S. C	12	30
Mobile, Miss	5.5	29
Galveston, Tex	9	30
Los Angeles Calif., (Inner harbor)	2	30
San Francisco, Calif., (Main channel)	32	40

ACKNOWLEDGMENTS

The writer is indebted to Capt. W. S. Crosley, U. S. N., Hydrographer, Navy Department, to Maj. Milo P. Fox, Corps of Engineers, U. S. A., and to R. L. Faris, M. Am. Soc. C. E., Acting Director, U. S. Coast and Geodetic Survey, for valuable assistance in the preparation of this paper.

DISCUSSION

C. L. HALL,* M. AM. SOC. C. E. (by letter).—The author has made what amounts to a survey of standard river and harbor practice. The special type of Mississippi River dredge described is not, and cannot be, used on the Upper Mississippi. The dredge used there is a pipe-line dredge, without cutter, and without propulsion machinery, and was first introduced in 1899.† The dredge is held by spuds, the position of the suction pipe being controlled by a catamaran,‡ moved by a swinging engine. The accordion type of joint, originally used, has been replaced by a ball joint. The normal cut of the dredge is 80 to 100 ft. in width.

The Upper Mississippi type of dredge is used not only for obtaining requisite channel depths, but also for securing material for wing-dams, the two purposes being combined whenever possible. The "sand dams" were first developed during the season of 1911.§ They are built out from shore, and usually are completed only to low-water elevation. They are then capped by the ordinary brush mattress with rock cover. The sand dam is such an economical type of construction that, when dredges are available, it has almost entirely replaced the brush and rock dam along the Upper Mississippi, except in stretches where the slope is abnormally steep.

DE WITT D. BARLOW,‖ ASSOC. M. AM. SOC. C. E. (by letter).—This very interesting paper contains two tables of costs, which the writer thinks are likely to create misunderstanding unless they are further explained.

Table 2 gives the average performance of Government-owned "Hydraulic Pipe-Line Dredges, Cutter Type" during the years 1923, 1924, and 1925, and states the costs as 8.32, 8.48 and 7.44 cents, respectively.

Table 3 ("Performance of Hydraulic Pipe Line Dredges Working Under Contract"), gives the average price per cubic yard for the years 1923, 1924, and 1925, as 20.99, 21.3, and 19.9 cents, respectively. While the casual reader might draw the inference that the cost of hydraulic dredging under contract is vastly greater than that done by Government plant, this is not the case.

In the first place, it will be noted that the total yardage on which the Government figures of approximately 8 cents are based, is about 1 500 000 per year, a very trifling amount; whereas the contract yardages, from which the average prices are obtained, vary between 13 800 000 and 25 000 000 per year. So there is no reason to suppose that there is any similarity between the work done by the Government and by contract plant. Even, however, if the work were identical, the figures would not be susceptible of comparison for the following reason. The contract prices represent the total cost to the United States of performing the amount of useful dredging done. The writer uses the words "useful dredging" advisedly. Contracts are let by the United States for the excavation of channels of fixed widths and depths, and with

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† *Engineering-Contracting*, Vol. XL, No. 13, p. 361 *et seq.*
‡ *Engineering News*, July 24, 1913.
§ *The Military Engineer*, September–October, 1924, p. 389.
‖ Pres. and Director, Atlantic, Gulf & Pacific Co., New York, N. Y.

certain fixed side slopes. The contract instrument provides that no payment will be made to the contractor for any work done outside the lines of the desired improvement. The contractor is required, however, to clear those lines, that is, to remove all material from within them. To accomplish that end, it is necessary to remove very large quantities of material from without the prescribed sections. This is called "no pay overdepth." It very often happens that in order to be paid for 1 000 000 yd. of excavation the contractor must remove 1 500 000 yd., or even 2 000 000 yd.

In the computation of costs of operating by Government plant, no account is taken of the amount of useful dredging performed. The amount of yardage set down as excavated is all the material removed by the dredge regardless of what it was. Even this determination is often made with the most trifling regard for accuracy. Sometimes the daily reports of the dredge captains are taken, which are no more than guesses, and are not likely to be too low.

The Government cost figures are obtained by dividing certain expenditures by the yardage thus determined. The expenditures themselves, however, do not include all, or nearly all, the cost to the United States of doing the work. They usually contain merely the operating cost of the plant while it is on the job. They never include plant rental, that is, interest and depreciation on the investment, which is a very considerable item; nor insurance (fire, marine, and liability); nor do they include any of the costs of maintaining idle equipment.

When the United States rents its hydraulic pipe-line dredges to private parties, it charges 0.1% per day of the first cost of the plant for rental, that figure having been determined after a careful study of their actual cost experience. That being the case, it is obvious that a dredge that costs $1 000 000 has fixed charges running against it in the amount of $1 000 per day, no account of which is taken in determining the nominal cost of Government operations. The upshot of the whole matter is, therefore, that the 8-cent costs are only costs in a very special sense. They are quite meaningless, unless one happens to know the formula. In order to compare these nominal Government costs with contract prices, it is necessary to make all the foregoing corrections, and others. Within the writer's experience the effect of these corrections is often to increase the cost to several hundred per cent. of the nominal one. It is astonishing what happens to a fraction when one continues to add to the numerator and to subtract from the denominator.

The writer can see no harm in keeping and publishing such nominal costs as long as the correlative circumstances controlling them are published or generally known. The difficulty is that they are not generally known and these data, which may be useful in the War Department, for purposes of its own comparisons, are likely, as in this case, to come before the public in an unexplained form and to mislead, although there is no intention on the part of any one to do so.

W. M. Black,* M. Am. Soc. C. E. (by letter).—Major Hall brings out two points which were omitted from the paper. The type of dredge described as

* Maj.-Gen., U. S. A. (*Retired*); Engr. (Black, McKenney & Stewart), Washington, D. C.

developed on the Mississippi River is efficient only for handling the very finely divided materials which form the bars in that river below the mouth of the Missouri, as well as in the Missouri, Arkansas, and other of the tributaries of the lower river. In the upper sections of the Mississippi River, above the mouth of the Missouri, the materials forming the bed of the stream are coarser and for their removal require the application of a greater cutting power than that afforded by the water jets of the lower-river type of dredge. The "sand dams" of the upper river mark an advance in the art of river improvement which should have been noted.

The comments of Mr. Barlow are also of value. To avoid misconceptions some further discussion of the costs of dredging seems desirable. Mr. Barlow is in error when he states:

"* * * that the total yardage on which the Government figures of approximately 8 cents are based, is about 1 500 000 per year, a very trifling amount; whereas the contract yardages, from which the average prices are obtained, vary between 13 800 000 and 25 000 000 per year".

A more careful scrutiny of Table 2 will show that the figures for "average performances" give the average of the work of 42 dredges in 1923 and of 43 dredges in 1924 and 1925, operating in approximately 20 different districts and having a total output varying between 61 000 000 and 72 000 000 cu. yd. per year.

With respect to the over-depth dredging, each contract makes provision for payment for the removal of materials from below the contract depth, usually for 1 ft. or 2 ft., in order to make due allowance for the unavoidable irregularity of the bottom of a cut. The statement that 1 500 000 or 2 000 000 cu. yd. "very often" must be removed in order to be paid for 1 000 000 cu. yd. of dredged material, may be true for exceptional cases. However, available records show that, as a rule, the yardage of "unpaid-for" over-depth dredging varies between zero and 30% of the yardage paid for. In certain cases dredging beyond over-depth limits is credited to the output of Government dredges, but the amount of such yardage is very small. Any extensive over-depth dredging by Government plant is done intentionally to avoid the necessity for early maintenance dredging.

The statement as to the method of computing the yardage excavated by dredges is in error. As a rule the yardage is computed from a comparison of surveys made before and after the dredging. These surveys are made by a field party, not connected with the dredging operation and reporting direct to the District Office. The same procedure is followed in measuring yardage in contract dredging.

The cost figures for the Government work include all operating expenses, except plant rental and depreciation, interest on investment, and fire, marine, and employers' liability insurance. The cost of the repairs and of plant maintenance is absorbed in the cost of the work.

The record for eleven years (1916 to 1926, inclusive) shows that the yearly average of losses to Government floating plant by fire, by collisions, and from miscellaneous causes is $164 195.47. The estimated value of the plant in

1925 was $47 654 706.00. The losses thus averaged $0.003446 per year per dollar of the investment.

The cost figures given for contract work in Table 3 do not show the entire cost of the work to the United States as they do not include the costs of surveys, of inspection, or of office overhead.

The charges for the rental of Government plant are made high purposely in order that such plant shall not be used by private individuals in competition with a contractor's plant. The illustration given by Mr. Barlow of the handicap formed by rental charges is misleading in that there is no Government pipe-line dredging unit which has cost $1 000 000. In very few cases is the valuation as high as $500 000.

It is the policy of the Engineer Department to have dredging done by contract unless there is too great a difference between the cost of contract work and the cost of work with Government plant. In order to be able to judge relative costs more clearly, since 1926 the cost accounting system has included rental and depreciation on all items of floating plant. In estimating the costs of dredging with Government plant in comparison with bids by contractors the Department takes into consideration interest on investment (on value of plant used), and fire, marine, and employers' liability insurance. These, however, are not included in the published cost figures as actual charges against the work.

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Paper No. 1702

THE DELAWARE RIVER FROM PHILADELPHIA TO THE SEA*

By F. C. Boggs,† M. Am. Soc. C. E.

With Discussion by Messrs. Charles M. Spofford, Henry J. Sherman, Earl I. Brown, Harry Taylor, F. C. Warner, William Gerig, and F. C. Boggs.

The Delaware River has its origin in a small lake in Southeastern New York. It flows in a general southerly direction, emptying into Delaware Bay, an estuary of the Atlantic Ocean, the entrance to which is marked by Cape Henlopen on the west and Cape May, the southern end of the New Jersey promontory, on the east. The upper reaches of the river have the usual characteristics of a mountain stream. Its middle section is less torrential, but is broken by reefs and rock ridges which, with its shallow depths, render it unfit for navigation except for small boats in the quiet pools between the rapids. At Trenton, N. J., the river begins its long meander as a tidal stream through the low coastal plain bordering the ocean. From Philadelphia, Pa., to the sea, the section with which this paper is concerned, it is a stream of varying depths, increasing in width from 5 000 ft. just below the Schuylkill River near Philadelphia, to approximately 4 miles where it enters the Bay.

The total length of the waterway from the Atlantic Ocean to its source is approximately 365 miles, of which the stretch comprising the first 131 miles from the sea to Trenton, is tidal.

Although there are numerous streams flowing into the Delaware River throughout its entire length, there are two only which can be dignified by the term tributary rivers. These are the Lehigh and the Schuylkill, each having its source in the Blue Ridge Mountains in Pennsylvania, the former

* Presented at the meeting of the Waterways Division, Philadelphia, Pa., October 5, 1926.

† Col., Corps of Engrs., U. S. Army; Dist. Engr., Philadelphia, Pa.

entering the main stream at Easton, 50 miles above Trenton, and the latter at the southern boundary of the City of Philadelphia.

The discharge of the Delaware River at its mouth is a combination of tidal and fresh-water flow. No data are available on which to base an exact determination of the fresh-water flow, but that it is a material factor is evidenced by tidal and current records at Liston Point, the section where the waterway designation is changed from river to bay. At this point the duration of ebb is 6.6 hours as against 5.8 hours for flood, while the ebb current at strength in a vertical section in the channel averages approximately 2.4 knots as compared with the flood of only 1.8. The combined discharge at this section during hours of ebb is at the rate of about 800 000 cu. ft. per sec., and a very rough approximation gives the fresh-water discharge during the same period as averaging nearly 250 000 cu. ft. per sec.

Original Condition

In its original condition, the Delaware River below Philadelphia was obstructed by a number of shoals or bars, over which the minimum usable depths at mean low water were between 17 and 22 ft. Fifty years ago, before the systematic improvement of the river was undertaken by the Federal Government, a ship sailing up the channel to Philadelphia required in some cases four days to make the trip, owing to the necessity of waiting for high water to pass over these shoals.

During the winter months the ship was also likely to be delayed a day or more by ice jams blocking the entire channel at the "Horse Shoe", where the river bends sharply to the westward at the lower end of Philadelphia Harbor. The harbor itself was obstructed by Smith and Windmill Islands and their adjacent longitudinal bars, and by shoals in its upper reaches.

River Bottom

Except at a few localities where there are rock outcrops, the bed of the Delaware River below Philadelphia is composed of sand and gravel overlaid in most sections by a light mud. At some places this overlying mud extends to considerable depths, rendering the construction of dikes and bulkheads difficult and costly. This is particularly true where these works are required to sustain the thrust of earth fills behind them.

Omitting the rock sections, dredging can readily be done by the suction type of dredge, although for contract work clam-shell and dipper dredges have been used almost exclusively for the removal of material from the channel.

At times muddy, the Delaware River below Philadelphia can not be classed as a silt-bearing stream, although in time of freshets in the upper river, or in tributary streams, some deposit is undoubtedly due to silt brought into the river by flood waters. Caving banks and the washing of the banks by waves are also unquestionably the sources of additional deposits.

The shoaling in the channel which must be removed to maintain the project depth is caused partly by these deposits, but the large bulk of such shoaling cannot be accounted for from these sources alone. The tendency of cross-currents to wash the material from the sides of the bed into the

channel has been noted by many investigators. The writer believes that the largest part of the dredging for channel maintenance is due to two factors: First, the building up by these cross-currents, and by deposition of suspended material, of a middle ground where, due to islands or to the peculiar bends in the river, the ebb flow and the flood flow do not naturally take the same channel; and, second, the gradual movement of the surface of the mud bottom toward the dredged channel, resulting in a shoaling of the channel and a general lowering of the bed of the stream to some distance on each side of the channel width. This latter condition is shown by a comparison of surveys made in 1909 and 1924.

Tides and Currents

A complete and exhaustive study of tides and currents in the Delaware River has recently been made by the United States Coast and Geodetic Survey.* A copy of this report was loaned to the writer by the courtesy of the late E. Lester Jones, M. Am. Soc. C. E., Director of the Survey, and what follows under this heading is based almost entirely on this report.

The oldest recorded tidal observations in the Delaware River Basin date back to about 1835. These early records are based on unimproved instruments and methods and are probably far from accurate. The introduction of the automatic, self-registering tide gauge in 1854 and the adoption of standard time about 1885 permitted a more nearly accurate record.

The tide from the open ocean enters Delaware Bay between Cape May on the New Jersey shore and Cape Henlopen in the State of Delaware, a width of about 10 nautical miles. The sweep of the tide follows the estuary of Delaware Bay which widens gradually from the Capes to a maximum width of 23 miles, this width decreasing to about 4 miles at the upper end of the Bay where it merges with no definite topographic feature into the Delaware River. Following the river the tidal wave moves over a generally, but in no case an abruptly, decreasing front until at Philadelphia the width of the river is about 2 000 ft., and at Trenton, the head of the tidal flow, is about 1 000 ft.

A study of the tides at Philadelphia was facilitated by the fact that there is available an almost continuous record from 1901 to 1920. These records are of particular value because they cover much of the period of channel improvement by dredging and the introduction of training works, and afford some opportunity to study the tidal regimen as affected thereby.

The mean lunitidal interval, that is, the interval of time between the transit of the moon and the following high or low water, is given for Philadelphia for high water as 1.5 hours for the first ten years and 1.47 for the second. For low water the corresponding figures are 9.01 and 8.94 hours, respectively. The periodic seasonal change is readily grasped from a study of the graph in Fig. 1.

The authors of the study believe the variations in the lunitidal interval from month to month are due to the variation in the frictional resistance

* "Tides and Currents in Delaware River and Bay," by L. M. Zeskind and E. A. Le Lacheur, U. S. Coast and Geodetic Survey *Special Publication No. 123*, 1926.

to the tidal wave caused by differences in river level, and they point out that in the summer months when the river levels are highest, the intervals are the least, while the reverse is true for the winter months.

The duration of the rise and the fall of the tide at Philadelphia is given in Table 1.

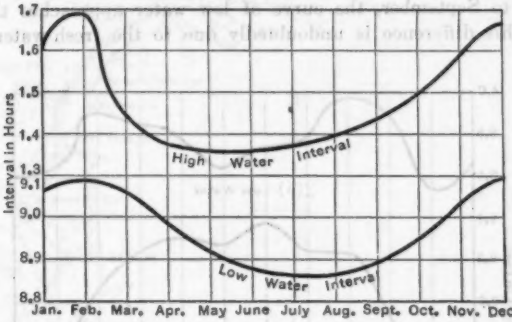

FIG. 1.—ANNUAL VARIATION IN LUNITIDAL INTERVALS, PHILADELPHIA, PA.

Table 1 is typical of river tides where the constant fresh-water flow opposes the flood and runs with the ebb, thus shortening the time of the former and increasing that of the latter. It is interesting to note that while for each full month the sum of the time of rise and fall equals 12.42 hours, a tidal cycle, yet daily records of individual tidal cycles show in some cases considerable variations, the time of an individual rise or fall being, in a few instances, one or two hours greater or less than the monthly mean, due to the effect of the wind advancing or retarding the tidal movement of the water.

TABLE 1.—DURATION OF RISE AND FALL OF TIDE, PHILADELPHIA, PA., ANNUAL VARIATION.

Month.	Duration of rise, in hours.	Duration of fall, in hours.	Month.	Duration of rise, in hours.	Duration of fall, in hours.
January	4.96	7.46	July	4.92	7.50
February	5.01	7.41	August	4.93	7.49
March	4.79	7.63	September	4.98	7.44
April	4.81	7.61	October	4.99	7.43
May	4.89	7.53	November	5.00	7.42
June	4.91	7.51	December	4.98	7.44

To the navigator, the characteristics of the tidal movement in which he is most interested are the low-water and the high-water levels. The annual variations in the plane of low water for Philadelphia for the period, 1901 to 1920, are shown in Fig. 2 (*a*), and corresponding variations in high water are shown in Fig. 2 (*b*). A study of these diagrams is interesting. It is seen that the annual variation in high-water level is more than twice that for the low-water level, and that while the former appears to run through

the year in a single cycle, the latter has two clearly marked cycles. Both are a minimum in February and while the maximum high is in June, yet the April high is very close to the June figure and corresponds with the maximum low which occurs in April. Here, however, the similarity of the curves ceases and while the high remains at nearly a constant maximum from April to September, the curve of low water approaches the minimum in July. This difference is undoubtedly due to the fresh-water flow of the river.

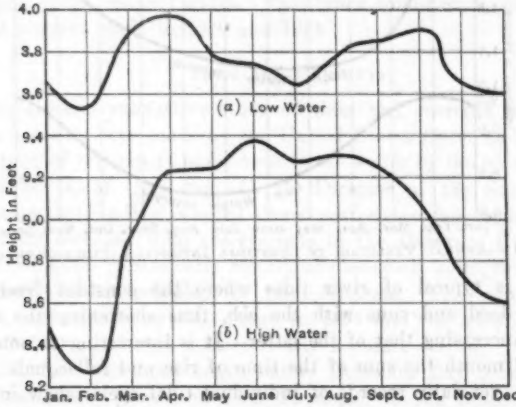

FIG. 2.—ANNUAL VARIATIONS IN LOW, AND IN HIGH WATER, AT PHILADELPHIA, PA.

The highest water recorded on the Philadelphia gauge was on October 11, 1903, when the river rose 4.22 ft. above mean high, or 9.45 ft. above the mean low, for the period, 1901 to 1920. The lowest record for the same period was on March 29, 1919, on which date the water fell 4.05 ft. below the mean low-water plane for that period. The mean range of tide for any year of the period may be obtained by subtracting the mean low for that year from the mean high for the same year. This range varies between 5.03 and 5.34 ft., averaging 5.23 ft. for the 20 years.

From a brief glance at the map of the Delaware River (Fig. 3), it is evident that northwest winds tend to drive the water out of the river, thus producing a tide lower than usual, while an east wind tends to drive the waters up the Bay and river, thus causing a high water higher than usual. Additional tables and discussions in the report from which the data contained herein were extracted, show these phenomena very clearly. An interesting case is cited in that report. During the blizzard in Philadelphia in March, 1914, the wind began blowing from the northwest on March 1 at the rate of 25 miles per hour and at 6:00 A. M., on March 2, it reached a velocity of 40 miles, which was maintained for a considerable time. As a result there was only one high water and one low water between 4:00 P. M., March 1, and 2:00 P. M., March 2, instead of the usual two high and two low, the water falling steadily during the entire 22 hours, dropping almost 4 ft. below normal.

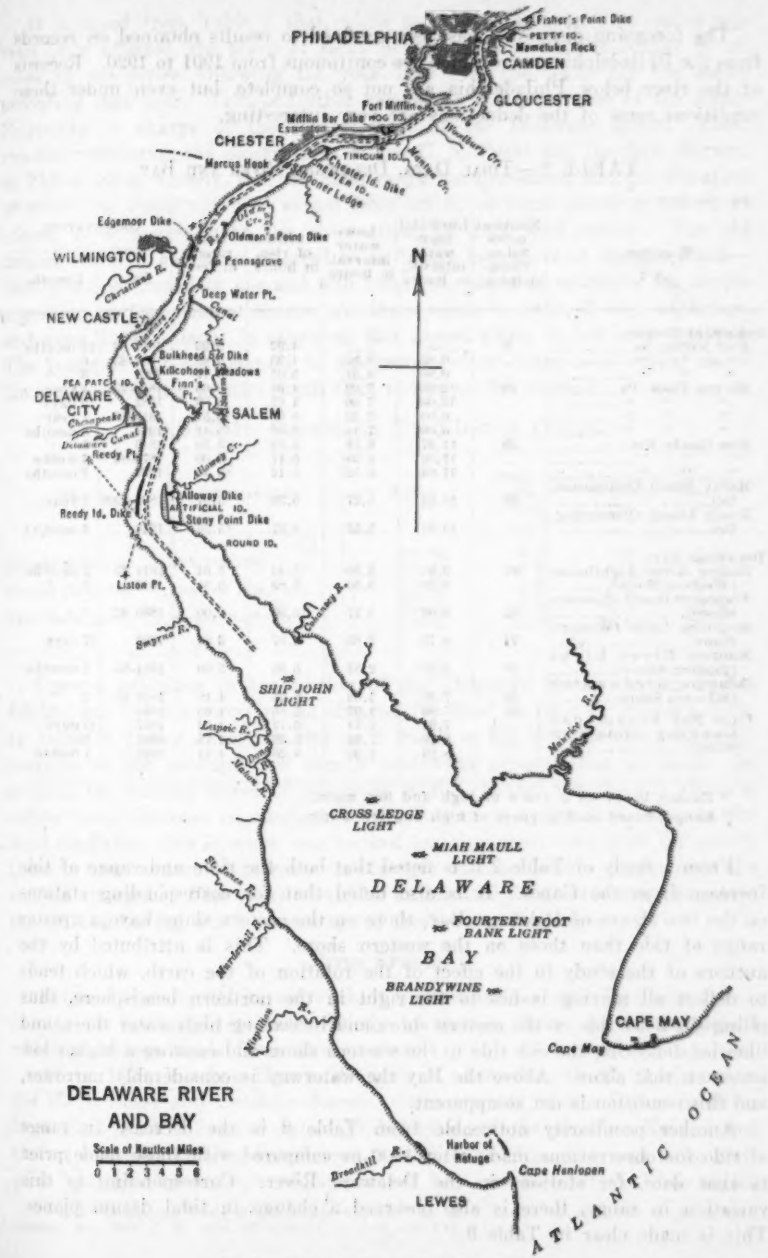

FIG. 3.—MAP OF DELAWARE RIVER AND BAY.

The foregoing discussion has been confined to results obtained on records from the Philadelphia gauge which are continuous from 1901 to 1920. Records of the river below Philadelphia are not so complete, but even under these conditions some of the deductions are very interesting.

TABLE 2.—TIDAL DATA, DELAWARE RIVER AND BAY.

Location.	Nautical miles below Philadelphia.	Lunitidal high-water interval, in hours.	Low-water interval, in hours.	Duration of rise, in hours.	Mean range, in feet.	Observation.	
						Date.	Length.
DELAWARE RIVER:							
Fort Mifflin, Pa	8	.....	....	4.92	5.93	1879-80	14 months
" " "	..	0.83	8.32	4.93	5.88	1882-83	6 "
" " "	..	0.96	8.37	5.01	*5.17	1924	3 "
Marcus Hook, Pa	18	12.25	7.29	4.96	6.06	1878-79	1 year
" " "	..	12.03	7.36	4.67	6.08	1881	10 days
" " "	..	0.09	7.42	5.09	5.51	1915-16	1 year
" " "	..	0.08	7.44	5.06	†5.41	1924	3 months
New Castle, Del	29	11.57	6.18	5.39	5.50	1840	5 "
" " "	..	11.56	6.39	5.17	6.00	1873-86	2 weeks
" " "	..	11.64	6.52	5.12	†5.42	1924	3 months
Reedy Island Quarantine, Del	39	10.65	5.27	5.38	5.91	1896-1900	4 years
Reedy Island Quarantine, Del	..	10.94	5.57	5.37	*5.56	1924	3 months
DELAWARE BAY:							
Mahon River Lighthouse (Western Shore)	61	9.94	3.50	6.44	5.64	1841-43	5 months
	..	9.39	3.50	5.89	5.31	1883	2 "
Fortescue Beach (Eastern Shore)	62	9.06	3.17	5.89	6.00	1880 82	1 "
Mispillion Light (Western Shore)	74	8.72	3.85	4.87	3.84	1883	17 days
Maurice River Light (Eastern Shore)	67	8.80	2.84	5.96	5.60	1884-85	2 months
Delaware Breakwater (Western Shore)	86	7.88	1.63	6.25	4.16	1863-68	1 "
Cape May Steamboat Landing (Eastern Shore)	82	7.83	1.97	5.86	4.65	1836	1 "
	..	7.91	1.74	6.17	4.74	1867	10 days
	..	8.29	1.91	6.38	4.78	1883	3 "
	..	8.16	1.90	6.26	4.71	1885	1 month

* Ranges based on 3 years of high and low water.

† Ranges based on 8½ years of high and low water.

From a study of Table 2 it is noted that both the time and range of tide increase from the Capes. It is also noted that for corresponding stations on the two shores of Delaware Bay, those on the eastern shore have a greater range of tide than those on the western shore. This is attributed by the authors of the study to the effect of the rotation of the earth, which tends to deflect all moving bodies to the right in the northern hemisphere, thus piling the flood tide on the eastern shore and increasing high water there, and likewise deflecting the ebb tide to the western shore and causing a higher low water on that shore. Above the Bay the waterway is considerably narrower, and this condition is not so apparent.

Another peculiarity noticeable from Table 2 is the decrease in range of tide for observations made after 1900 as compared with those made prior to that date, for stations in the Delaware River. Corresponding to this variation in range, there is also observed a change in tidal datum planes. This is made clear in Table 3.

It is noted from Table 3 that, while the plane of mean high water has been little affected, the plane of mean low water is materially higher in all cases in the series following the year 1900 over the plane for the series preceding that year. This is further verified by the experience of the Army Engineers in charge of the improvement of the Delaware River. Their records corroborate the observations of the U. S. Coast and Geodetic Survey, at Philadelphia, namely, that the tidal range has decreased, and the elevation of mean low water has been raised from 0.5 ft. at some points to 0.8 ft. at others, as compared with the observations of 1880 and earlier. The old datum, however, generally referred to as "mean low water at Marcus Hook—1879", has continued in use and still forms the plane of reference for depths appearing on Government charts. As there would be little, if any, advantage and some disadvantages in changing this datum plane, it has been retained. The result is not detrimental to navigators, since there is at actual mean low water a slightly greater depth than shown on the charts.*

TABLE 3.—CHANGES IN TIDAL DATUM PLANES.

Station.	Referred to.	Series.	Mean tide level, in feet.	Mean high water, in feet.	Mean low water, in feet.
Philadelphia, Pa.	Staff of 1901	1891-92	6.07	8.91	3.23
		1901-20	6.36	8.96	3.76
Marcus Hook, Pa.	Staff of 1878	1878-79	3.12	6.10	0.14
		1915-16	3.42	6.17	0.67
New Castle, Del.	Bench Mark No. 4	1840-86	13.06	16.04	10.08
		1916-24	13.70	16.42	10.99

Current velocities in the section of the Delaware River between Philadelphia and the sea vary in the channel for flood at time of strength from 1½ knots in Lower Delaware Bay to 2 knots in the river proper. Channel currents at ebb strength are usually somewhat greater than at flood. In general, the vertical curve of velocities in the channel for the ebb shows a rather rapid decrease in strength as the bottom is approached, whereas for flood conditions this is much less marked and at some points near the mouth there is even a decided increase in strength at considerable depths over the surface strength. This is as would be anticipated in a river emptying into the sea and having a fair percentage of fresh-water discharge.

RIVER STUDIES

Before discussing the actual work of improvements on the Delaware River, it may be interesting to refer briefly to certain theoretical investigations which have been made from time to time.

About 1878, the late Henry Mitchell, M. Am. Soc. C. E., Assistant in the U. S. Coast and Geodetic Survey, made a study of the Delaware River from Fort Mifflin, 8 miles below Philadelphia, to the mouth. He followed this at a somewhat later date by other studies based on new and more complete hydrographic surveys which had not been available at the time of his

* See "Tides and Their Engineering Aspects," by G. T. Rude, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 92 (1928), p. 606 and Fig. 21.

original report.* As a result of his first study, Mr. Mitchell laid down the following rules for this section of the river:

First.—The transverse section is directly proportional to the discharge.
Second.—The width is proportional to the discharge.
Third.—The mean depth is the same in all sections.

Mr. Mitchell pointed out as the most interesting feature of these studies the "constancy of mean depth". It is not intended by this statement, nor by Mr. Mitchell's third rule, to imply that mean depths were actually the same in all sections, as at various points there were some variations due chiefly to inequalities of the soil. However, by plotting mean depths along the length of the river, he shows that a horizontal straight line at a depth of 18.64 ft. would be the correct rectification of the irregular curve of actual mean depths.

Starting with this assumption of constancy of mean depth, the width of the river becomes the independent variable. Actual widths at various sections were plotted, and a rectified curve of widths using the plotted points as guides was drawn. Mr. Mitchell found that this curve corresponded to a parabolic formula. The theoretical cross-sections are obtained by multiplying theoretical widths by the mean depth of 18.64 ft.

The practical application of these studies is stated by Mr. Mitchell as follows:

"The persistent tendency to constancy of mean depth throughout the whole length of the estuary would seem to discourage the hope of improvement by dredging. * * * Except for two or three shallow reaches of short extent, there is plenty of water from the ocean to Philadelphia, and at these obstructed reaches, artificial contraction of the waterway may be made without altering the course of the stream or sensibly reducing the tidal volume; so that deepening may be induced where required without changing the conditions elsewhere".

Further studies were made in 1882 by Mr. E. A. Giesler, Assistant Engineer, under the Corps of Engineers, U. S. A. He called attention to the fact that while there was a definite increase in cross-sectional areas going toward the mouth, yet the increase was not a steady one, but was subject to frequent and irregular fluctuations which he very pertinently stated might be due to local causes. He felt that these local fluctuations might confuse the result, and, therefore, in order to eliminate them, he used instead of the actual cross-section at any given point, the mean of ten cross-sectional areas, five above and five below the section under consideration. He also based his theoretical smooth curve of increasing cross-sections on three sections at widely separated intervals of distance on account of their position on the more uniform parts of the curve developed by using mean cross-sections obtained as just described. From his study Mr. Giesler determined that the curve showing the increase in cross-sectional area of the river from Philadelphia to the mouth approached very closely an hyperbola.

* U. S. Coast and Geodetic Survey Report, Appendix No. 8, 1883, and Appendix No. 12, 1885.

Frank C. Warner, M. Am. Soc. C. E., Assistant Engineer in the Engineer Department of the Army, in 1917, modified Mr. Giesler's figures by using the mean of three cross-sections above and three below the section considered. He adopted an hyperbolic formula for increases in sectional areas similar to that used by Mr. Giesler. Mr. Warner states:

"The practical application of the theoretical law thus obtained must be made only in connection with studies of local conditions, or otherwise the results may be erroneous. Applied to a straight tidal river channel with uniform conditions of bed and banks throughout its whole length, definite and reliable results may be expected, but when applied to the Delaware River channel, in which variable conditions exist, such as sharp bends and bed and banks composed of materials varying from soft mud to sand, gravel, boulders, and even ledge rock, it is apparent that such theoretical law can be applied only as a check, but reliable in so far as it will determine limits in regulation beyond which it will not be safe to go with due regard to the regimen of the river as a whole".

H. F. Flynn, M. Am. Soc. C. E., who was Assistant Engineer on the work on the Delaware River, in 1918 called attention to the fact that Mr. Warner's results were based on a survey made in 1916, after a considerable amount of work had been done, including dredging and dike construction. He believed that the natural regimen of the river should be used instead of an artificial state. He noted also that investigators following Mr. Mitchell had used three sections at widely separated points on the river for determining their curves of areas, and he objected to this method for the reason that such sections do not "necessarily indicate that the river has a proper cross-section at those points, but rather that the relation of width to area happens to be such that the river is able or rather forced to maintain them."

In his study, Mr. Flynn went back to the cross-sectional areas and widths used by Mr. Mitchell, based on the survey made about 1880 before the natural state of the river had been materially affected by artificial works. He then, by the method of least squares, developed the following formula for the theoretical curve of increasing cross-sections:

$$X = \frac{a - b\,y}{y + c}$$

in which,

a, b, and c are constants;

X represents the cross-section of the river at any point, in hundreds of square feet; and,

Y represents the distance along the center line of the channel from the place of origin, in nautical miles.

Table 4 gives the results obtained from the formulas developed by these investigators and Fig. 4 shows the relation between the theoretical curves of increasing cross-sections resulting therefrom.

For cross-sections Mr. Flynn adopted the statement by Mitchell that in straight reaches a sine curve coincides very closely with the natural bottom. From this hypothesis the following formula was derived,

$$W = \frac{\pi}{2}\frac{A}{D}$$

in which,

D = maximum depth, in feet.
A = area of cross-section, in square feet.
W = surface width, in feet.

TABLE 4.—THEORETICAL AREAS OF CROSS-SECTION AT HALF TIDE.

Distance below Philadelphia, in nautical miles.	Square Feet.		
	Mitchell.	Warner.	Flynn.
5	100 000	104 800	106 800
10	114 000	119 900	124 200
15	137 000	138 300	145 200
20	170 000	161 100	171 200
25	213 000	190 100	204 000
30	264 000	228 400	246 700
35	325 000	281 100	304 700
40	396 000	358 300	387 900
45	476 000	.....	517 400
50	565 000	.....	746 500

Mr. Flynn adds:

"It is recognized that there are many elements affecting the width of the river that have been neglected in the above method of location of training dikes. The effect of curvature has not been taken up, but it is not large. The matter of the location of the deepest part of the channel has been disregarded. It is believed, however, that the method may be used to advantage. It is better adapted to the upper reaches than to the broader portions below. In the narrow, upper portions of the river the dredged portion of the channel forms a considerable fraction of the total width between banks. Here the contraction required to make the channel self-maintaining, that is, to make the distance between the banks such that it will correspond to a natural depth of 38 ft. at half tide, is comparatively small. Proceeding down stream, however, conditions change, until at the lower end we find that the width of the dredged channel is a very small fraction of the total width, and the dredged channel assumes more the form of a ditch or canal dug through the comparatively flat river bed. Any attempt to create a natural depth of 38 ft. at half tide would require so considerable a reduction in width between banks as to be impracticable."

In an interesting paper on the "Relation of Depth to Curvature in Channels", H. C. Ripley, M. Am. Soc. C. E., states, under the heading, "Conclusions":*

"Professor Mitchell has shown,† in his investigations of the estuary of the Delaware River embracing a length of 52 miles from Philadelphia, Pa., down 'that the mean depth is the same in all sections and is equal to 18.64 ft. from all soundings for 46 nautical miles * * *'.

"Now, as the maximum depth in the straight reaches of any stream is equal to the mean depth multiplied by a constant, and the maximum depth in bends is equal to the mean depth multiplied by the same constant plus the effect due to curvature, it follows that the maximum depth in all straight reaches will be the same. Any attempt, therefore, to deepen the channel

* *Transactions*, Am. Soc. C. E., Vol. 90 (June, 1927), p. 238.
† U. S. Coast and Geodetic Survey Report, 1883, pp. 239–245.

by dredging will disturb this persistent tendency to uniformity of depth. Hence, it would appear that it is not practicable to secure any permanent improvement in the navigable channel of that river by dredging. It is evident, therefore, that the logical method would be to convert the straight reaches into channels of suitable curvature by means of training walls and to let Nature do the rest. In this way equality of mean depth would be preserved and the increased depth secured would remain permanent forever."

Further reference to these theoretical studies will be made subsequently.

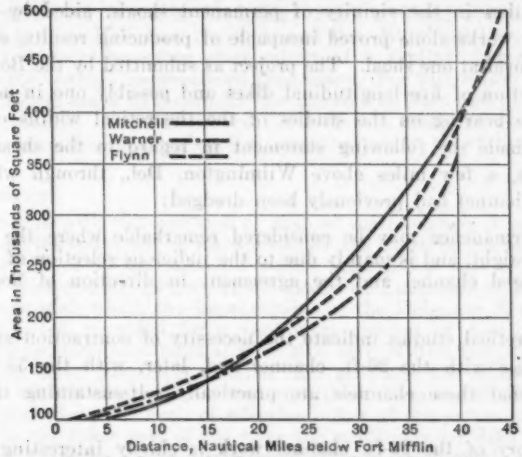

FIG. 4.—CURVES OF THEORETICAL CROSS-SECTION.

PROJECTS

The first project for the systematic improvement of the Delaware River was adopted in 1885. Previous to that year, and commencing as early as 1836, Congress had appropriated funds and work had been done on isolated improvements, such as ice piers and the dredging of channels across the more obstructive shoals.

The River and Harbor Act of July 5, 1885, however, carried an appropriation for the "improvement of Delaware River from Trenton, N. J., to its mouth", and a project was accordingly prepared by a Board of Engineers appointed to consider the subject. This project provided for the construction of a channel having a least depth of 26 ft. at mean low water, and a least width of 600 ft., to extend from the Port Richmond grain piers, in Philadelphia Harbor, to deep water in the Bay, a distance of about 60 miles. This project was prepared a short time after the studies made by Mr. Mitchell and Mr. Giesler, and it is interesting to note the influence that these studies may have had on the proposed work. The late Col. William H. Heuer, U. S. A. (*Retired*), M. Am. Soc. C. E. (then Major, Corps of Engineers, U. S. A.), in his preliminary examination refers to Mr. Mitchell's study, and Mr. Giesler's investigations form a part of Major Heuer's report. Major Heuer states:

"The remedy for improving the channelway is contraction, so as to compel both inflowing and outflowing tides to occupy the same channel".

While the report of the Board of Engineers which reviewed Major Heuer's report, and on which the adopted project was based, made no direct reference to the studies by Mitchell and Giesler, they were undoubtedly in mind and possibly used in the location of some of the dikes recommended.

The improvement was in general to be secured by the construction of low-water dikes in the vicinity of permanent shoals, aided by dredging if the training works alone proved incapable of producing results, supplemented by rock removal at one shoal. The project as submitted by the Board involved the construction of five longitudinal dikes and possibly one in addition. As having some bearing on the studies of the theoretical widths of the river, the Board made the following statement in regard to the shoal at Cherry Island Flats, a few miles above Wilmington, Del., through which a self-sustaining channel had previously been dredged:

"This permanence may be considered remarkable where the river is so wide and straight, and is mainly due to the judicious selection of the location of the dredged channel and the agreement in direction of flood and ebb currents."

The theoretical studies indicate the necessity of contraction at this place, but experience with the 26-ft. channel and, later, with the 35-ft. channel, has shown that these channels are practically self-sustaining through this shoal.

The history of the 26-ft. channel work is chiefly interesting as demonstrating the ineffectiveness of low-water dikes, and the fact that even after raising these works to high water, dredging was necessary to produce the desired depth in any reasonable time. In justice to the original conception it should be added that once the depth was obtained by dredging, the dikes assisted in maintaining it, and it may be that the failure of the contracting works in the first instance was due to character of material.

Before the completion of the 26-ft. project the growth of Philadelphia as a port, and the general change from sailing vessels to steam with increased draft, had awakened a demand for a deeper channel. In 1899, Congress authorized a new project superseding the one of 1885, this new project providing for a channel 30 ft. deep and 600 ft. wide from Philadelphia to deep water in the Bay, at an estimated cost of $5 810 000.

When the 30-ft. project was adopted, about $1 600 000 had been expended on the 26-ft. channel, which depth had been secured over widths varying from 200 to 600 ft. from Philadelphia to the Bay, except for uncompleted lengths aggregating about 4 miles. In addition, the harbor at Philadelphia had been improved by the removal of Smith and Windmill Islands, by the establishment of pierhead and bulkhead lines, and by the partial removal of Mameluke Rock (Fig. 3), an area which interfered with the movement of vessels in the upper harbor. The report on which the 30-ft. project was based was submitted by a Board of Engineers, which had before it a preliminary study

made by the late C. W. Raymond, Brig.-Gen., U. S. A. (*Retired*), M. Am. Soc. C. E. (then Lieutenant Colonel, Corps of Engineers, U. S. A.). In this study Colonel Raymond stated:

"The greatest depth which could probably be maintained at reasonable cost might perhaps be determined by a study of the river itself, if the form and dimensions of the channel were solely the result of the action of flowing water, but this is not the case. Natural obstructions, immovable by water, such as the rock at Schooner Ledge, exist in some parts of the channel; and in other places where bars have persistently formed, old wrecks and other artificial obstructions have been found by dredging. Such artificial obstructions have been removed at Five Mile Bar, Smith Island Bar, Bulkhead Bar, and Reedy Island Bar, and doubtless other obstructions of a similar character will be discovered as the channel improvement progresses. For this reason the various attempts heretofore made to determine the theoretical form and dimensions of an improved channel in the Delaware River from a study of existing cross-sections have been found after careful examination to be of little practical value."

He also states:

"In connection with this question of channel maintenance, it may be asked whether the depths could not be maintained by the construction of dikes properly located to direct the currents and confine their action within the limits of the channel. The original project of the Board of Engineers of 1885, in general accordance with which the improvement of the Delaware River has been thus far conducted, provided for a very large amount of dike construction. Dikes have accordingly been built at Five-Mile Bar, Mifflin Bar, and Bulkhead Bar, and they have admirably fulfilled the purposes for which they were designed. It will be observed, however, that these dikes are all located in the part of the river above the limit of effective inflow of the waters of the marine tide wave. In the lower part of the river the conditions for dike construction are much less favorable. The dikes would have to be of great length. They would be difficult and very expensive to construct, on account of the softness of the bottom, and they would have to be located with the greatest skill and care to avoid obstructing the inflow of the flood tide. Owing to the action of ice and storms these structures would require large expenditures for annual repairs. Moreover, it has been found practically impossible to locate and construct such works on the lower part of the river on account of the real or fancied grievances of riparian owners. Accordingly, the Board of Engineers of 1896 recommended the abandonment of dike construction in this part of the river and the improvement of the channel by dredging. When the Board of Engineers of 1885 submitted its report, the cost of dredging was relatively so great as to amply justify the expense of dike construction. With the improved dredging machinery and low prices of the present time the construction of dikes is no longer generally considered advisable for the improvement of a bar within the mouth of a tidal river."

The Board recommended as a part of the project a longitudinal bulkhead near the Pennsylvania shore, just above the Christiana River, and also one near the New Jersey shore nearly opposite Reedy Island (Fig. 3). These bulkheads were to serve two purposes: First, to act as training dikes; and, second, to form basins into which material dredged from the channel could be deposited. The latter dike finally took the form of an enclosure in the river which has since been raised by those deposits into an island very properly known as Artificial Island (Fig. 3). The bulkhead above the Christiana River was started, but was not completed. The chief matter of interest during the work

on the 30-ft. project was the introduction of the sea-going hopper dredge. This will be described later.

The demands for a channel to accommodate the deeper draft vessels required by the economics of ocean-borne commerce, again became insistent, and an investigation was made with a view to deepening the channel to Philadelphia to 35 ft. The result was the existing project which was adopted in 1910. It provides for the formation of a channel 35 ft. deep from Philadelphia to deep water in Delaware Bay, a distance of about 63 miles, 1 000 ft. wide through Philadelphia Harbor, 1 000 ft. wide at bends, except one quite sharp one at Bulkhead Bar which is to be 1 200 ft., and 800 ft. wide in straight portions. The original estimated cost of the project was $10 920 000, with $300 000 annually for maintenance. It may be noted that General Marshall, the then Chief of Engineers, U. S. Army, was of the opinion that the estimated cost of maintenance was far less than what he considered would be the result of actual work. Experience has demonstrated that General Marshall's opinion was well grounded. In 1922, a re-estimate of the cost of the completed work and that still remaining to be done on the original project was made, and the new figure, $15 300 000, was approved. While this amount is 50% greater than the estimate of 1910, it is noted that the higher cost of labor and materials since that date would more than account for this increase. The cost of maintenance, however, has increased beyond any difference chargeable to higher labor and material costs.

The District Engineer, Major Herbert Deakyne (now Brig.-Gen. Deakyne, Assistant Chief of Engineers, U. S. A., M. Am. Soc. C. E.), who submitted the report reviewed by the Board of Engineers for Rivers and Harbors, stated that past experience indicated that the maintenance of a deep channel by dredging alone would be unduly expensive, unscientific, and border on the impracticable. He added that "systematic contraction and control of the tidal flow are the proper means for the maintenance of the channel." And, further, "it is not expected that these controlling works will obviate the necessity of dredging a channel, but it is expected that they will simplify the problem of maintenance." He recommended construction of certain dikes for the purpose of contraction or controlling the currents.

The Board of Engineers in submitting the report on which the present project is based, agreed with the District Engineer in his recommendations.

The work on the 35-ft. channel necessary to obtain the first deepening from 30 ft. has been carried on practically without interruption since 1910, this original work having been done almost wholly under the contract method. The entire channel from deep water to Philadelphia has been completed with the exception of a 2-mile stretch in the upper end of Philadelphia Harbor, where dredging is now in progress, a few small rock areas in the channel near Chester, Pa., which it is hoped will be removed at an early date, and the removal of Mameluke Rock in the Upper Philadelphia Harbor, which is the chief obstacle to the completion of the work. About 90 000 cu. yd. of this ledge still remain, although it is noted that a 35-ft. channel of reduced width can be dredged to the westward of this area.

On the dikes recommended, the following work has been done (see Fig. 3):

Fisher Point Dike, completed to within 300 ft. of Petty Island. Full completion of this dike was prevented by action of citizens of New Jersey who objected to the cutting off of the channel between Petty Island and the New Jersey shore.

Mifflin Dike, partly completed. Work on this dike was interrupted by the construction of Hog Island Shipyard and objections to the closing of the channel north of Tinicum Island.

Chester Island Dike, from the north end of Chester Island to the New Jersey shore, completed.

Oldman's Point Spur Dike, completed.

Bulkhead Bar Spur Dike, completed.

Edgemoor Dike, small portion finished.

Reedy Island Dike, partly completed. Work stopped due to protests of riparian owners.

Artificial Island, completed.

For maintenance, contracts are let for such work as is necessary in Philadelphia Harbor, and the dredging in the lower river is carried on by Government-owned and operated plant consisting of three sea-going hopper dredges and two pipe-line disposal dredges. At present (October, 1926), a navigable channel 35 ft. deep at mean low water and of widths varying from 200 ft. to the full width of the channel, extends from above the congested portion of Philadelphia Harbor to deep water in the Bay, except at a few isolated spots where the filling in of the channel has kept ahead of the dredging work. On the last examination made in August, 1926, the controlling depth on the center line of the channel was 32.7 ft., this, however, being at one point only.

Each year, however, shows an improvement in maintenance work and there is every reason to believe that with the erection of additional dikes as funds will permit for the combined purpose of disposal basins and control of the current, the maintenance of the channel at project depth can readily be held by dredging.

Methods of Work

As previously stated, the improvement on the Delaware River is a combination of dredging and dike construction. For the latter work the only features of note are that low-water dikes have proven ineffective, and that, due to ice and storm conditions and the nature of the bottom, these dikes must be of a considerably more substantial cross-section than was originally anticipated, this being still further accentuated when the dikes are also required to withstand thrust from back-fill.

Experience in dredging has shown the great advantage of the sea-going hopper dredge, particularly in maintenance work, over the stationary type. The critical feature in the dredging work is the disposal of the dredged material. Several methods have been tried. At first, the material was deposited in available locations in the river outside the channel. This was not found to be economical for the reasons that the bulk of the material quickly found its way back into the channel and had to be re-dredged; the available deposit areas were very limited due to the draft required for the dredges in going to and from the areas; and the filling of these areas frequently interfered with the

use of side channels for tugs, barges, and small freight carriers of shallower draft.

The method of agitation was then tried. This consisted in dredging by the sea-going dredges for long continuous periods, until the hoppers were filled with solid material, the lighter mud and sand being permitted to flow overboard. This was done only on ebb tide, with the expectation that the solids in the overflow would remain in suspension and be carried away by the tidal currents. This also proved uneconomical since these solids did not remain in suspension for any length of time, but came to rest in the channel below the point of operation, thus requiring re-dredging.

The method then adopted is that now in use, and consists in depositing the dredged material on fast land behind embankments or in the river in areas surrounded by dikes. The operation consists in dredging with the sea-going hopper dredges, generally between 30 and 60 min., until the economic load is obtained, this being determined from a consideration of material in hoppers, time of dredging, and time required to move to disposal basins. With bucket dredges the material is loaded into bottom-dump barges which are towed to the place of deposit. At the deposit basin, the dredgings from the sea-going dredges or the dump barges are deposited in basins dug in the bottom of the river and are then pumped into the prepared area by a suction pipe-line dredge. Incidentally, these areas, built up as described to 12 ft. and more above the plane of mean low water, may form valuable sites for industrial and other purposes.

The bucket dredge and the suction pipe-line dredge are familiar to all engineers. The sea-going hopper dredge is not so common. Essentially, it is a sea-going vessel, self-propelled, which contains, in addition to the quarters for the crew and the propelling machinery, suction pumps and bottom-dumping bins of a capacity depending on the size of the vessel. The suction pipe, so arranged as to permit raising and lowering, is fitted at its outer end with a drag head, consisting in general of a heavy casting with grated opening on the bottom. In operation, the suction pipe is lowered below the water level, the pump started, and the pipe then lowered until the drag head touches the bottom. As the dredge proceeds slowly over the area to be dredged, the material from the bottom, combined with a certain quantity of water, is pumped into the hoppers in the dredge. When the economic load has been obtained, the suction line is raised, the pump stopped, and the dredge proceeds under its own steam to the deposit basin, where the load is dumped. In brief, the sea-going hopper dredge is a combined pumping, towing, and dump-scow plant in one hull. Figs. 5 and 6 are views of two of these dredges.

The cost of the work done with the hopper dredge varies with the type of material and the distance from the deposit basins. For sand and gravel combined, with a comparatively long haul, the present cost is approximately 14.4 cents per cu. yd. of material removed. For soft material, with a comparatively short haul, the figure is 2.8 cents. The cost of pumping the material ashore by the pipe-line dredge varies from 2.5 to 10.8 cents per cu. yd., depending on the cost of construction of deposit basins and length of pipe line.

FIG. 5.—VIEW OF DREDGE *New Orleans*.

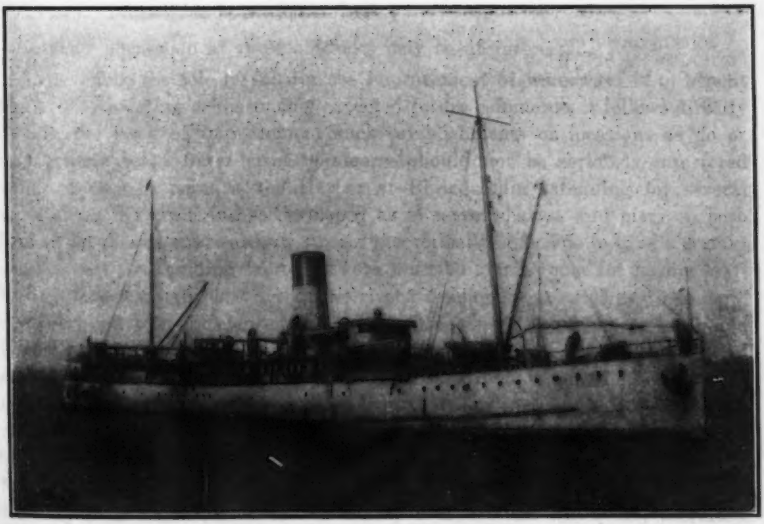

FIG. 6.—VIEW OF DREDGE *Manhattan*.

Depth by Contraction

From a study of the work done under the various projects, and having in mind the studies of theoretical widths which have been discussed, the query naturally arises, why was greater use not made of the contraction system?

As a matter of fact the improvement of the river since the first systematic project for 26-ft. depth was started, has been by dredging supplemented at certain critical sections by longitudinal dikes or by spur-dikes.

Practically, however, the use of dikes has not proven so simple as a theoretical discussion of the problem would indicate, and the number constructed or projected are not in line with those that would be required under any conclusion based on such theoretical discussion, nor such as are desirable for the purpose of economic maintenance.

Some of the obstacles which have seemed and still seem unsurmountable are:

(*a*) Impracticability of placing such structures where they might not interfere with the use of existing or prospective commercial developments on shore, or where shore owners might not object to being cut off from deep water.
(*b*) Uncertainty under the present state of the science as to just where dikes should be located.
(*c*) The time required before any definite improvement throughout the entire length of stream could be obtained.
(*d*) The fixing of the channel at a certain depth.
(*e*) The increased difficulty of navigation of the channel by large vessels if it were developed on a series of curves instead of tangent ranges.
(*f*) Limited funds available.

A brief discussion of these obstacles may be of interest.

(*a*).—Since the sole reason for the improvement of waterways is to benefit and develop existing or reasonably expected future commerce, it follows directly that in the work of improvement such establishments or locations as do or may contribute to water-borne commerce should not be seriously interfered with. A case in point is the Harbor of Philadelphia extending for several miles along the river and so developed as to serve wharves and piers on both banks. In this case, a width of stream theoretically sufficient to give a certain depth is not the deciding factor. There must be ample room for harbor traffic both longitudinally and across the river. Anchorage grounds are necessary, and there must be sufficient width to permit ocean-going vessels to enter and leave their slips without too great interference with other vessels or risk of endangering structures on the other side of the stream. On the other hand, if the necessary width for this consideration is less or greater than the theoretical width for the desired depth, it is inconceivable that owners of existing wharves and bulkheads would re-arrange them to comply with lines based on this theoretical width. Structures in Philadelphia Harbor designed and located solely for reducing the stream to a theoretical width are, therefore, out of the question.

It is admitted that this is an extreme case, and proponents of the training-dike system of improvement may claim that, while true for Philadelphia Harbor, it would not be true on the length of river below Philadelphia. The channels on the two sides of Tinicum Island and the stretch of the river below the Island (Fig. 3) present a waterway of too great a section for the development or maintenance of a channel of the present project depth if dependence is placed on any of the theoretical formulas. In order that the river currents alone might maintain this channel, a longitudinal training wall should be constructed from fast land to Tinicum Island, cutting off the back channel near the Pennsylvania shore, with probably a dike extending below the Island. In fact, the former dike was recommended under one of the original projects of lesser depth than the present one. Incidentally, there would also be required for the theoretical width necessary for maintenance in case the back channel were blocked off, the removal by dredging of a large part of the Island since the material of which it is formed is too heavy and compact to be moved by the river currents.

On this back channel is located one of the large plants of the Westinghouse Electric Company, and, while this industry might consent to the change providing it owns the riparian rights and that portion of the Island within an extension of its property lines, as this would permit it to have access to deep water, yet it is extremely doubtful whether the Town of Essington and the yacht clubs just below would be so complaisant. Likewise, one might imagine the attitude of the Baldwin Locomotive Works, located on the Pennsylvania shore just below Tinicum Island, if it were to find its present wharf, to which it has dredged a channel at its own expense, left several hundred feet landward of a training dike.

This is not an isolated case. The Pennsylvania and Delaware shores are lined for almost the entire distance from Philadelphia to below Wilmington by cities and manufacturing establishments the influence of which would undoubtedly be brought to bear against any system of works that would cut them off by a practically currentless stretch of water from the main channel.

That this objection is real and not fancied is evidenced by past experience. At least two dikes designed for the purpose of contracting the channel were stopped before completion, due to the protests of riparian owners.

(*b*).—In discussing the studies of the river, it has been noted that no two investigators agree as to the theoretical formulas for the contraction required to produce a certain depth of channel. In practice, one must proceed with caution; the cut-and-try method must be adopted. River engineers are familiar with the fact that any artificial work may cause serious changes in the regimen of a stream above and below the structure in question, this effect sometimes extending for long distances. Temporary structures or parts of the proposed work must be built and time must then be permitted to elapse before a knowledge of results sufficiently definite to warrant further progress can be obtained.

(*c*).—Due to the necessity of proceeding slowly and also on account of limitation in available funds, many years would elapse before a complete system of dikes could be built. In the meantime, while certain sections of the

river might be deepened to the desired draft, others would be untouched. The navigability of a channel depends on the governing depth of that channel. There might be a 35-ft. depth throughout 59 of the 60 miles of waterway from Philadelphia to the sea, but a depth of 20 or 25 ft. for the remaining mile or even a small fraction of it, would limit the size of vessels capable of navigating the channel to what could be carried over this governing section. In other words, there would be little real practical improvement until the entire system was completed.

(*d*).—Assuming longitudinal dikes could be placed with full assurance that the pre-desired depth could be obtained, there is the risk of stabilizing that depth, or of being compelled to construct a new series of contracting works to obtain a greater depth. Suppose, for example, the first project depth for the Delaware River had been obtained by the construction of longitudinal dikes. It is problematic whether funds would have been forthcoming for the 30-ft., and then the 35-ft. channel, if dependence had been placed on contraction works alone, for it must be remembered that theoretically the channel would develop close to the dikes along curved sections and the cost of constructing new dikes in the deeper waters would be much greater than the original construction in comparatively shallow water.

(*e*).—It is a fact that a sinuous channel could be navigated, but it is equally true that such a channel would be much more difficult for the handling of large boats than one laid out as a series of straight stretches for which easily distinguishable range signals might be erected, with short turns marked by light buoys or some other distinctive marks. The dangers of collision or of grounding in night running on a 100-mile channel, such as that first mentioned, would almost prohibit its use during hours of darkness, and would tend to diminish its usefulness even during daylight hours. The tendency of the channel to deepen on the outer part of the curve and hence near the contraction works would also increase the danger of navigation.

(*f*).—The revenues of even the Federal Government are limited. These revenues must cover a multitude of necessary governmental expenditures. The result is that the amount available for all the river and harbor work throughout the United States is and always has been comparatively small. Shall these funds be used for works which after a lapse of many years may ultimately be beneficial, but which give little if any relief in the meantime; or shall they be used in such a way as to maintain what is already available and also give a progressively deeper channel, thereby permitting a gradual but steady increase in the size of cargo carriers? It would seem that the answer is obvious.

The writer has attempted to touch very briefly the obstacles to a complete control of a stream like the Delaware River by a system of dikes alone. He does not want to be misunderstood and be set down as opposing all use of dikes for partial control. That these works will not produce the desired deepening of a channel in the Delaware River in any reasonable time has been proven on that river by past experience, for in every case where they have been constructed, it has been necessary to expedite the action of the current by recourse to dredging. Past experience has been equally positive in proving that prop-

erly located dikes will assist materially in maintaining the channel. One principle of location of these dikes has been fully exemplified, that is, that they should be placed so that the ebb and the flood tides shall be forced into the same channel. Wherever this has been done the benefits, as regards maintenance, have been very apparent.

There are many places on the river where work of this kind could be done without injury to commerce or to riparian owners, and with great benefit to the cost of maintenance, and ultimately these works will be constructed. In the meantime, dredging must be carried on continuously to maintain the depth of channel already obtained, and basins must be constructed in which to place the material dredged. In this connection, it is again noted that the dikes and bulkheads constructed for these basins are designed and located not only for this purpose but with a view to their effect on controlling the currents which it is anticipated will result in a material decrease in maintenance dredging.

As is generally true, there is a happy medium between extremes which all things considered gives the best results, and the Government engineers have attempted to strike this medium, between the extremes of "all dikes with no dredging" and "all dredging with no dikes," in so far as funds and the legitimate demands of commerce for early improvement and continuous maintenance of increased depths will permit.

Economics

A paper on the Delaware River would be only partly complete without some reference to the commercial and economic side of the question.

The Port of Philadelphia includes the City of Philadelphia with its wharves on the Delaware and Schuylkill Rivers, Camden, N. J., Gloucester, N. J., and some minor points north and south of Philadelphia. This port serves a large area. For Eastern Pennsylvania, Southern and Central New Jersey, and Delaware, involving a population of about 5 000 000, rail freight rates to Philadelphia are equal to or lower than similar rates to any other first-class seaport. It also has competitive rates with other ports for a large area, including Western Pennsylvania, Ohio, Indiana, Illinois, Michigan, Wisconsin, Iowa, Minnesota, and the Dakotas.

By far the largest percentage of the water-borne tonnage is handled over the wharves of the City of Philadelphia. Practically all these wharves on the Delaware River are served by a belt line which permits shipments or receipts by or from any of the three railroad companies serving the community, the Pennsylvania, the Reading, and the Baltimore and Ohio. The Delaware waterfront is also paralleled by a wide avenue permitting easy trucking between piers and to cross streets leading to all parts of the city. On the Schuylkill River, while there is no belt line, a large majority of the wharves have switch connections with one or more of the railroads mentioned.

These railroad companies have large water terminal developments, including coal and ore piers with mechanical unloaders, general cargo piers, and grain elevators. Of the latter, one has a capacity of 1 250 000 bushels, another of 2 250 000, and a third is now (1926) under construction, the first unit of which, with a capacity of 2 500 000 bushels, will be in operation at an early date.

For many years the City of Philadelphia has followed the policy of development of its water-front. Since 1909, thirteen modern piers have been constructed by the municipality. Figs. 7 and 8 show cross-sections of two of the latest types of pier.

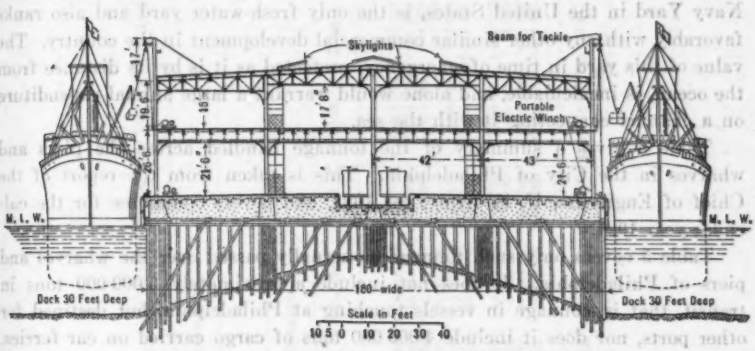

FIG. 7.—TYPICAL CROSS-SECTION, SOUTHWARK PIERS, PHILADELPHIA, PA.

Aside from the rail terminals and the city-built piers, there is little permanent wharf equipment for handling ship's freight, dependence being placed in general on the cargo booms and winches of the vessels themselves.

Under the policy adopted by the City, none of its municipally owned wharves is operated by it, but is leased to shipping lines. The writer regrets that at least a few of these facilities have not been reserved for municipal operation, where all comers could be sure of obtaining berths and reasonable loading, unloading, and storage rates, although it may be added that these rates as charged at privately owned or leased wharves compare favorably with similar rates at other ports.

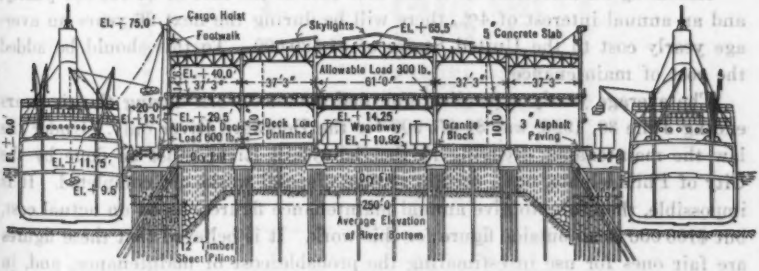

FIG. 8.—SECTIONAL ELEVATION, MCKEAN STREET PIER, PHILADELPHIA, PA.

There are three shipyards on the Delaware River in and near Philadelphia, which can construct or repair any vessel that can navigate the channel.

A large part of the inbound tonnage in the Port of Philadelphia consists of crude oil and oil products. Oil refineries and tank farms are located on the Schuylkill River and on the Delaware River at Marcus Hook, Pa., with one

large plant on the New Jersey side about 10 miles south of Camden and one just north of that city.

In addition to this commercial development, the Delaware River channel serves the League Island Navy Yard, which, in addition to being the largest Navy Yard in the United States, is the only fresh-water yard and also ranks favorably with any other similar commercial development in the country. The value of this yard in time of emergency, protected as it is by its distance from the ocean, is incalculable, and alone would warrant a large annual expenditure on a channel connecting it with the sea.

Table 5 gives a summary of the tonnage handled across the piers and wharves in the City of Philadelphia. This is taken from the report of the Chief of Engineers, U. S. Army, for 1925, and covers commerce for the calendar year 1924.

Table 5 covers only such tonnage as actually passed over the wharves and piers of Philadelphia. It does not include approximately 5 000 000 tons in transit, that is, tonnage in vessels touching at Philadelphia, but destined for other ports, nor does it include 4 000 000 tons of cargo carried on car ferries. The tonnages for Camden, N. J., and Wilmington are also excluded from this summary, although ships for these ports make use of the channel between Philadelphia and the sea throughout its entire length for the former and over approximately two-thirds of its length for the latter.

Every engineering work is subject to scrutiny as to whether or not it is a paying proposition. Examination of the Delaware River, from this point of view, will show that the total cost of all new work on this river between Philadelphia and the sea, including the deep-draft channel for the lower six miles in the Schuylkill River, not only on the present project, but also on all former ones, for work done by the Federal Government, to June 30, 1925, was $26 000 000 in round numbers.

Assuming an amortization of the first cost at the rate of $1 000 000 yearly, and an annual interest of 4%, there will be during the next 26 years an average yearly cost to the United States of $1 520 000. To this should be added the cost of maintenance.

The average yearly expenditure on maintenance work during the six years ending June 30, 1926, was $1 475 000 for the Delaware River only. Under the law the maintenance dredging in the Schuylkill River must be done by the City of Philadelphia, until certain sewage disposal works are completed. It is impossible, therefore, to give annual maintenance figures, based on actual cost, but $100 000 is an outside figure for this work. It is believed that these figures are fair ones for use in estimating the probable cost of maintenance, and, in fact, are somewhat excessive, as they include the cost of disposal basins, which will be available for several years.

These amounts, interest on cost of new work, and yearly maintenance, give $3 095 000 as the amount the United States Government will be required to pay each year for the next 26 years on account of the 35-ft. channel from Philadelphia to the sea.

TABLE 5.—PORT OF PHILADELPHIA (THE DELAWARE AND SCHUYLKILL RIVERS), NET COMMERCE 1924.

Classes of commodities.	Foreign.				Domestic.						Total.	
	Imports.		Exports.		Coastwise.				Other Domestic.			
					Receipts.		Shipments.					
	Tons.	Value.	Tons.	Value.	Tons.	Value.	Tons.	Value.	Tons.	Value.	Tons.	Value.
Animals and animal products	38 933	$ 12 122 637	11 272	$ 3 871 429	14 198	$ 12 911 941	11 555	$ 8 826 838	4 789	$ 7 953 564	80 747	$ 45 686 409
Vegetable food products	1 261 264	97 280 700	1 171 142	57 759 301	83 948	17 771 835	138 541	25 346 059	259 792	13 594 441	2 907 687	211 752 336
Other vegetable products	43 131	12 952 967	23 999	3 030 985	33 632	7 053 024	9 462	6 117 549	4 205	374 424	114 429	29 528 969
Textiles	76 562	38 235 512	11 366	2 688 283	86 709	87 630 945	40 655	32 868 396	.....	.....	215 292	161 438 136
Wood and paper	266 704	12 346 592	15 508	1 609 778	384 708	7 072 945	*14 986	5 692 603	81 158	994 549	718 064	27 716 467
Non-metallic minerals	1 184 514	10 959 537	1 358 679	46 010 766	6 200 976	106 832 618	1 364 730	24 230 008	8 175 511	58 339 644	18 279 410	246 372 568
Ores, metals, and manufactures of	599 596	8 028 741	61 323	6 349 115	31 921	2 614 187	326 077	24 323 902	41 316	2 045 437	1 060 233	43 361 382
Machinery and vehicles	1 915	998 300	28 258	6 194 860	2 720	894 340	15 229	5 465 571	679	421 250	48 801	13 969 321
Chemicals	117 921	5 940 188	84 346	6 029 882	81 076	3 582 135	54 377	11 460 613	358 943	17 561 421	696 563	44 574 234
Unclassified	2 248	3 842 175	4 248	985 316	89 240	16 511 071	426 374	93 478 051	806 280	85 281 409	11 328 385	200 048 022
Total	3 592 788	$202 723 364	2 760 186	$184 474 715	7 009 128	$262 875 041	2 401 886	$237 809 585	9 675 673	$186 566 139	25 439 611	$1 024 447 844

What is the return on this yearly outlay? From Table 5, the figure of 25 000 000 tons is obtained as the total commerce passing over the wharves and piers in the City of Philadelphia. It would be erroneous, of course, to attribute this entire tonnage to the work done on the channel by the United States Government, since, before any work was started, there was an available depth of possibly 18 ft. at mean low water. The increase of tonnage due to increased depth as obtained by work done by the United States is impossible of exact determination, but it is believed a rough figure can be obtained.

Table 6 gives the number of vessel trips made during the year 1924, together with vessel drafts. The figures in Column (3) give the ratios between tonnage of cargo carried in a vessel of the draft given to that carried by a vessel drawing 18 ft. This is, as must be realized, quite arbitrary and may be far from the actual ratio. However, data were not available to determine the correct figures and the best obtainable were used.

TABLE 6.—Draft of Vessels and Number of Vessel Trips.

Draft of vessel, in feet. (1)	Number. (2)	Ratio of cargo to 18-ft. draft. (3)	Product. (4)	Total. (5)
More than 30	18	5	90	.....
28 to 30	129	4	516	.....
26 to 28	512	3	1 536	.....
24 to 26	363	2.75	998	.....
22 to 24	476	2.33	1 110	.....
20 to 22	664	1.75	1 162	.....
18 to 20	1 336	1.25	1 670	7 082
16 to 18	1 500	1	1 500	.....
Less than 16	5 798	0.75	4 348	5 848

Column (4) gives the product of Column (2) by Column (3). It is noted that, by using the arbitrary ratios adopted, the number of vessels of more than 18 ft. would carry 7 082 18-ft. vessel cargoes, while those under 18 ft. would carry 5 848. These vessels drawing more than 18 ft., therefore, carry about 55% of the yearly tonnage. Fifty-five per cent. of 25 000 000 tons gives 14 000 000 tons per year as the approximate increase of commerce due to the deepening of the channel by the Federal Government.

Using this tonnage and $3 000 000 as the annual cost to the United States, it is found that, for each ton, the United States is expending 21.4 cents or, since the distance from Philadelphia to deep water in Delaware Bay is 60 miles, 3.6 mills per ton-mile.

While there is much controversy in regard to the advisability of improving inland rivers, the writer has yet to hear any arguments against the development of waterways leading to large seaports. The Port of Philadelphia is claimed by some writers to be the second seaport in the United States in the amount and value of its water-borne commerce. Whether or not actually second, it is beyond argument that this port ranks well up in the list of those following the Port of New York.

Foreign imports and exports to and from Philadelphia carry about the same rate as those for New York and Baltimore. Rail freight rates to Philadelphia from these two cities for Class 6 commodities are 18½ and 17½ cents per 100 lb., respectively. Without its waterway the people of Philadelphia and its vicinity would be placed under a tremendous handicap in transferring imports and exports and its coastwise shipments by rail to or from either New York or Baltimore.

DISCUSSION

CHARLES M. SPOFFORD,* M. AM. SOC. C. E.—This paper impresses the speaker as a most enlightening contribution to engineering literature relating to the regulation of rivers. The feature of greatest interest to the speaker is the theory on the "constancy of mean depth" originally suggested by the late Henry Mitchell, M. Am. Soc. C. E., and adopted by various members of the U. S. Engineer Corps who have studied the river.

The author states that no two investigators agree as to the theoretical formulas for the contraction required to produce a certain depth of channel; that there still seem to be insurmountable obstacles in determining where dikes should be located; and that, in practice, it is necessary to adopt the "cut and try" process and to permit considerable time to elapse before sufficient knowledge of results can be obtained to warrant further progress. These conditions make pertinent the inquiry as to why small scale experiments on models of the river have not been made to determine the validity of the theoretical conclusions before spending large sums in construction. The money required for experimental tests would seem to be so insignificant compared with the cost of unsatisfactory construction that such means of investigation should be ignored only for the most potent reasons.

Experimental work of this character has been conducted for many years in German universities. Apparently, it has been well worth while, judging by the fact that in recent years several additional important laboratories, equipped to make such experiments, have been constructed and many experimental investigations made on German rivers.

The speaker inspected two of these European laboratories in May, 1926, the State Laboratory in Vienna, Austria, and the laboratory at the Technische Hochschule at Karlsruhe, Germany, in both of which he found much activity. At Karlsruhe, experimental work was being conducted under Professor Rehbock on the River Maas, in The Netherlands, and on the Quadälquiver, in Spain, and the speaker understood that experimental investigations had also been made in this laboratory on certain South American rivers. In recent years, a large and expensively equipped laboratory has been constructed in Stockholm, Sweden, and complete plans have been prepared for an extensive laboratory for this purpose at the Technische Hochschule at Zurich. Evidently, other countries than Germany are alive to the wisdom of such experimental investigations. Complete accounts of the experiments that have been performed in the various European laboratories have been published.† In this country, no such laboratory as yet exists, but the Massachusetts Institute of Technology is seriously considering the construction of one.

The figures given for the cost of dredging seem exceptionally low and it would be interesting to know whether these figures include all charges, such as overhead, interest on investment, salaries of engineers and officers con-

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† "Die Wasserbaulaboratorien Europas."

nected with the work, depreciation, insurance, and other factors. The question arises, if dredging by Government plant is done along the lower reaches of the river, why is it not also economical in Philadelphia Harbor where dredging is now done by contract? Possibly this is due to the fact that, while constant dredging is required in the lower reaches of the river, the dredging in Philadelphia Harbor is only intermittent.

In the paper, the construction of dikes, both as a means for controlling the channel and also for holding the dredged material within given areas, is mentioned. It would be of added value if information was presented as to the method of constructing these dikes; particularly as to whether wooden bulkheads have been used. It would seem, because of the comparatively soft material of the bottom at some locations, that the construction of dikes would be decidedly expensive.

Considerable attention is given to the commercial advantages resulting from the improvement of the Delaware River, thus establishing Philadelphia as a first-class port. The Port of Philadelphia is one of a notable group situated on rivers and canals, including such important ones as London, Hamburg, Antwerp, Montreal, New Orleans, Baltimore, Manchester, and Houston. These ports, all of which are some distance from the sea, and many of which have required large expenditures for improvements, are among the busiest in the world. Apparently, inland ports are in favor with shippers, but just how far the Government should go in the development of river ports in a country where natural harbors of deep draft, requiring but moderate expenditures for maintenance and improvements, already exist, is difficult to determine. Why should the Government spend large sums in improving the Port of Philadelphia when considerable freight passing through this port could equally well be shipped from Boston, for example, which is at least one day nearer Europe and which possesses an admirable natural harbor and excellent shipping facilities?

It is true that, in time of war, interior ports are safer against destruction by enemy battleships, although they are not much better protected against airplanes. On the other hand, they are more easily obstructed by sunken vessels and are sometimes greatly hampered by ice. Even a port as far south as Baltimore was seriously hampered by ice in 1918. If private capital is to be used in developing a port, no one can justly protest. By the expenditure of large sums of private capital a self-sustaining port has been developed at Manchester, England, all ships entering it proceeding directly past the Liverpool docks. Even in this case, one naturally questions whether, if the same sum had been expended in Liverpool and upon the railways connecting Liverpool and Manchester, it would not have been for the economic advantage of the country.

HENRY J. SHERMAN,* M. AM. SOC. C. E.—A cross-current passing over a shoal area and thence across a channel deposits its burden of material, carried in suspension, into the latter. No doubt in the Delaware River, as else-

* Cons. Engr., New Jersey Board of Commerce and Nav.; Cons. Engr. (Sherman & Sleeper), Camden, N. J.

where, this is one of the principal causes of shoaling. Movement of a soft bottom in shallow waters, caused by passing vessels in narrow channels, is a constant source of shoaling. It is fortunate that there is now available complete and exhaustive studies of the tides and currents from a high source like the U. S. Coast and Geodetic Survey, on which to base future operations.

In Table 1, showing monthly duration of rise and fall of tides, it is observed that the greatest variation is 0.22 hour for rise, or 4.50%, while it is only 0.12 hour for fall, or 1.60%, the monthly average being decidedly constant, despite the great variation in single cycles noted by the author.

A further study of Fig. 2, showing annual variations in high and low water, reveals the maximum difference of 0.4 ft. for low water and 1.10 ft. for high water, the latter being nearly three times the former.

In Table 2, the differences in the tidal ranges on the two sides of Delaware Bay are so great as to be worthy of mention, being no less than 1.76 ft. at Maurice River, a large percentage of the total range.

In Table 3, it is shown that high water in New Castle, Del., has been raised 0.38 ft. as compared with only 0.05 ft. and 0.07 ft. at Philadelphia, Pa., and Marcus Hook, Pa., respectively, and that the low-water plane at Philadelphia and Marcus Hook has been raised exactly the same amount, namely, 0.53 ft. Opinions will probably differ as to the causes of raising the low-water plane, a number believing that the contraction of the river is the answer. The figures given for cost of dredging seem very low in comparison with contract work, even after allowing for contractor's profit, and marine and liability insurance, items which, it is understood, are not included in these figures.

The fact that the 26 and 35-ft. channels have been self-sustaining near Cherry Island, without reduction in the width of the river, as theoretical studies had indicated would be necessary, is a vindication of a conservative policy in dike construction. The summary of the obstacles preventing dike construction is excellent. Specific mention might also be made of the high cost of installation and maintenance of such structures.

The author speaks of the strong opposition of shore interests cut off by a dike, and particular reference was made to citizens of New Jersey stopping the completion of Fisher's Point Dike. In this connection it may be said, there is now a demand for the removal of the outer portions of that dike and the re-opening of the channel back of Petty Island. In September, 1926, the War Department held a hearing to obtain views on proposed harbor lines at this point. This channel would accommodate prospective commerce in the upper part of the City of Camden. When the dike was proposed many years ago, the riparian lands were of little value; the navigation interests cut off were small; not much interest was shown in the project; and no public body was charged with the duty of studying the effect of the jetty on State or local interests. To-day, however, with the great increase in the value of riparian lands, and State and municipal bodies watchful of all water-front changes, such strong opposition would develop against the building of any dike that the project would probably be abandoned. Opposition is so strong

to another dike that is partly completed that a provision was actually inserted in the River and Harbor Bill prohibiting the use of any of the funds for the continuance of that dike.

Under Obstacle (*c*), the author states: "The navigability of a channel depends on the governing depth of that channel". This is now 32.7 ft., so near the project depth, that further dike construction might appropriately be undertaken if this were the only obstacle.

Under Obstacles (*d*) and (*e*), it is observed that where longitudinal dikes have been constructed, the channel in every case is maintained in straight ranges and, in some cases, is a considerable distance from the dikes.

The author makes timely reference to the control of some piers by the municipality. Manufacturing and small shippers who cannot afford to locate on an expensive water-front should, nevertheless, have those facilities and be assured access to a pier at all times at reasonable rates.

Under "Economics" the interest rate has been taken at 4.50%, which appears somewhat high. A rate of 4% or even 3.5% as an average of Government borrowing over a long term of years might be fairer.

The author has ingeniously worked out a cost of 21.4 cents per ton to the United States for the increased tonnage due to the deepening of the channel beyond the actual depth of 18 ft. While, undoubtedly, this figure is high compared with Northern rivers, it probably would compare favorably with the alluvial streams of the South.

While the paper is directed specifically to the part of the Delaware River from Philadelphia to the sea, reference might be made to the proposed ship canal across New Jersey from Bordentown to Raritan Bay, as the estimated tonnage from this source will utilize a large portion of the river under discussion. The State of New Jersey, believing the United States is almost ready to undertake construction, has made an appropriation to begin purchase of the right of way.

The author has compiled the important data and engineering opinions of the best minds in the U. S. Engineer Corps and Staff bearing on the problem of regulating the Delaware River. Supplementing this are his own observations and conclusions on certain features of the work. His presentation of the arguments for and against dike construction is eminently fair and unbiased. Out of the controversy which has been waged on this question, a middle course has been adopted utilizing each method to some extent. Careful thought and study have been given to the expenditure of the funds committed to the War Department, and the results achieved justify the confidence reposed in those charged with the work.

In his plain and comprehensive review of the work accomplished and analysis of the problems involved, he has made a splendid contribution to the literature of river and harbor improvement.

EARL I. BROWN,* M. AM. SOC. C. E.—The author is to be congratulated upon the very complete justification he has given of the policy of placing more

* Col., Corps of Engrs., U. S. A.; Engr., Eighth Corps Area, Fort Sam Houston, Tex.

dependence upon dredging to secure immediate and positive increases in depth of channel than upon regulation by training walls and dikes. Not that walls and dikes should be neglected, but they should be constructed only as a supplement to dredging, after an exhaustive study of commercial requirements as to channel depths, and of the susceptibility of the river to improvement by regulation. Some rivers lend themselves to it more readily than others, the character of the bottom, the shape of the bed, and the amplitude of the tide being factors.

Many engineers, not themselves engaged in active practice of river improvement, but reading in current engineering literature of the extensive and expensive regulating works constructed in the tidal rivers and harbors of the Old World, and comparing them with the scarcity of such works in practice in the United States and the greater reliance here placed on dredging for these improvements, are tempted occasionally to protest that river engineers in this country are not utilizing regulating works as they should.

The tide and its resultant phenomena are the means whereby the regulating work in a tidal stream achieves its ends. Compare the usual ranges of 15 to 20 ft. in European seaports with the average range of 4 to 5 ft. in ports of the United States, and it will be appreciated that this country could not expect to accomplish the same result with the same structure. A study of tidal conditions and manifestations is not a matter of such vital importance to America.

It is not to be understood, however, that it is the speaker's purpose to minimize the importance of tidal studies in connection with the improvement of rivers. He believes, on the contrary, that it could and should be carried out to a greater extent than it usually is. If the results of a full and complete tidal study are available, the engineer can prepare his project and execute his work with a greater degree of confidence in the ensuance of benefit to navigation.

That part of the paper relating to studies made upon physics and hydraulics of the river does not indicate that much attention has been paid to these very important questions. However, it must be conceded that any improvement project must be based on a study of them if the work is to be accomplished in the most successful manner. Failure to plan the improvement so as to take full advantage of the knowledge derived from those studies is a step in the dark, which may be injurious.

Previous investigators only went so far as to endeavor to find a law of variation in the width and cross-section of the river, based on an assumption that the mean depth is the same in all sections. These investigators proceeded on empirical rather than theoretical lines, and found equations of either the parabolic or the hyperbolic type. It is now generally conceded by river engineers that the law of variation of width of a tidal river is best expressed by a logarithmic equation. Recent investigations show that there is a definite relation between the variation of the energy of the tidal wave and the width of a tidal river. If the bottom be horizontal, as reported by Mitchell and assumed by the other investigators, the equation is of the simple exponential

form as a function of the distance from the mouth of the river. If the bottom has a horizontal slope, the law of variation is more complicated, and is in the form of a compound exponential equation.

In most estuaries and tidal rivers, the amplitude, the speed of the wave, that is, its rate of propagation, and the velocity of the currents diminish more or less rapidly going up stream, because of the expenditure of the energy of the wave as a pure loss in overcoming the friction that results from the insufficiency of the mean depths and from the excess width of the bed, as well as from obstacles which it meets. By lessening these obstacles, it is possible not only to provide for the utilization of that energy for the conservation of the amplitude, the speed, and the velocity of the wave, but also to measure them to some extent by suitable means. All these factors have a bearing on the cost and probable degree of success of any project, and they cannot be neglected without paying the penalty in increased improvement and maintenance costs.

The Artificial Island mentioned by the author is an example. The curves showing the range of tides in the Delaware River as given in Fig. 4 emphasize a marked falling off of range of tide in the period 1890-1900 on that part of the river between Woodland Beach and near the upper tidal limit. Colonel Boggs indicates this falling off, but gives no explanation of it. However, by reference to the same source quoted by the author,* a statement is found that this change is to be attributed to the improvements effected in the river under the projects in force.

This explanation is not satisfactory, and cannot be accepted. The dredging of shoal areas beneath the plane of low water would have no effect on the height of tides, and the volume of material excavated from above that plane would not be sufficient, quantitatively, to account for the decreased volume of tidal flow above Woodland Beach. Some other explanation must be found.

The drop in range occurs in the reach between Woodland Beach and Delaware City, Del. Some of the most extensive contraction works on the Delaware River have been constructed in this reach. These include Artificial Island and the dikes connecting it with the mainland, and a dike running down stream from Reedy Island.

In his study of the law of variation of widths, H. F. Flynn, M. Am. Soc. C. E., gives the data shown in Table 7.

It will be noted that the greatest contraction of area is at Mile 35, near the lower end of Artificial Island, where, expressed in percentages, the original channel was about 5% in excess of theoretical requirements. It has been contracted to 75% of its original width, and has only 80% of its theoretically required width.

A contraction of this magnitude in the bed of a tidal river, the bottom of which is mobile, would cause scouring to the extent that the cross-section might be thereby restored approximately to its original size, and no material damage would be done in the long run to the tidal propagation. However, the bed of the Delaware River at this point cannot be classed as mobile. It is a very hard stiff mud which, when dug out by a bucket dredge, retains its

* *Special Publication No. 123*, U. S. Coast and Geodetic Survey, 1926, pp. 43 and 47.

shape and consistency. The river, therefore, has not been able to erode its bed and restore the magnitude of its cross-section. Consequently, the loss of energy due to this contraction is still felt in a decreased tidal range above the point of contraction. The injurious effects of this great contraction could be largely obviated, and the cross-section restored to a great degree by the removal of the two dikes which connect the island with the mainland.

TABLE 7.—CROSS-SECTION AREAS, DELAWARE RIVER.

Mile.	Mean area of section in 1881–82, in square feet.	Measured area in 1916, in square feet.	Theoretical area, as computed by formula, in square feet.	Decrease in area, 1881 to 1916, in square feet.	Present deficiency in area, in square feet.
31	248 700	241 100	256 000	7 600	14 900
32	263 200	248 000	267 000	15 200	19 000
33	301 400	236 000	279 200	65 400	43 200
34	324 500	249 000	291 500	75 500	42 500
35	321 700	241 000	304 700	80 700	63 700
36	317 300	261 000	318 900	53 300	57 900
37	331 600	323 800	334 100	7 800	10 300

It may be contended that this reduction in tidal range manifests itself, as it usually does, in a raising of the plane of mean low water in the upper section of the river, thus rendering increased depths available at that stage for navigation. This is a pertinent reply, and the resultant benefit must be balanced against those that would follow the increased tidal range, with its greater current velocity, and hence greater facility for self-maintenance.

It has been predicted by some engineers that a permanent improvement of the Delaware River by dredging could not be successfully accomplished, as there was a tendency to constancy of mean depth throughout the estuary. The author quotes the late Henry Mitchell, M. Am. Soc. C. E., to that effect, and also refers to the recommendation of H. C. Ripley, M. Am. Soc. C. E., that instead of dredging the straight reaches be changed into channels of suitable curvature by means of training walls to secure depths suitable to navigation needs.

The very satisfactory results that have been obtained by dredging are sufficient refutation of adverse predictions on that score, and Colonel Boggs indicates the many serious practical objections to a sinuous channel. From a theoretical point of view the increase in constant mean depth thereby obtained, will result in increased amplitude, speed, and velocity of the tidal wave, thus enabling the river to maintain the increased depth, when once obtained, by reason of the improved tidal conditions. There is, however, a limit beyond which tidal conditions can not be improved, and artificial maintenance will have to suffice. That limit, however, is not known.

As between straight and sinuous tidal channels, theoretical as well as practical considerations lead to a preference for the former. There is less friction and, consequently, less loss of energy in a straight than in a sinuous channel, and it is the preservation of the tidal energy to the farthest limit up stream that is to be sought. The principles enunciated by M. Fargue relating to

curvature and depth of channel, apply to streams flowing in one direction, but not at all satisfactorily to tidal streams. One of the greatest bends on the Delaware River, that just below New Castle, Del., is not accompanied by any perceptible deepening, and the channel lies on the convex point, not in the concave bend.

The author has touched upon shoaling as a result of lack of coincidence in certain places of the thread of the flood and ebb currents. It would have been very interesting and instructive if he had given a statement or tabulation showing the points where there is such a lack of coincidence, and had indicated the comparative amount and cost of maintenance dredging per mile at such places as compared with the same amounts in channels more suitably conditioned. It is the general impression that the larger part of the annual maintenance dredging is now done on a few bad places, perhaps not to exceed six. If that is a fact, it indicates the necessity of such a tidal study as herein referred to with a view to finding means of eliminating those conditions.

Summing up, the speaker's views are, that the general policy under which the improvement of the Delaware River is being done, is correct, and is based on sound principles, although some errors of judgment have been made in the locations of channels and of regulating works (including disposal grounds when in the bed of the river). A detailed study of the conditions of tidal propagation in the river might have prevented some of these errors, and might now enable them to be corrected, with a strong probability that maintenance costs could thereby be lessened, and future errors largely avoided.

HARRY TAYLOR,* M. AM. SOC. C. E.—The question of the relative cost of Government and contract dredging has been raised. The difference in cost of the two methods is explained at least partly by the fact that the Government owns comparatively few dredges and these are busy all the year. If work could be let in large contracts, which would keep all dredging contractors' plant busy for long periods and the contractors could be sure that when one job was finished another job would be available, so that all their plant would be always busy, the work should cost less; but whether the price to the Government would be less is another question, for experience has shown that low prices are obtained when there is little work and strong competition. However, such a happy condition for the contractor can never exist, for no contractor would be satisfied with the amount of work he had, and every one would be building more plant in the hope of getting more work and making a little more money.

Another item that favors the Government plant is that the Government does not have to go out and look for work, and is, therefore, not under the expense of bidding for it. Every contractor must bid for every job he obtains, and every bid costs money whether or not a job is secured. Not every contractor obtains work every time he bids, but he must charge up as part of his overhead expense the cost of preparing his bids, and some job must pay for it, otherwise the contractor will soon go out of business. Still another item is the expense of moving plant from one point to another. These

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items explain, in part at least, why the Government should be able to do work at a lower price than the contractor. Other items which add to the contractor's cost are depreciation and insurance. Depreciation is fully covered by the present cost-keeping methods of the U. S. Engineer Department, but owing to the very excellent care taken of all Government dredges, they last for many years and in some cases are still at work after their entire cost has been charged off. Therefore, the item of depreciation, when such plant is used, is a book charge only, and does not represent actual outlay of Government money. An example may be shown by the dredge, *Manhattan* (Fig. 6), which was built in 1908 for use in digging the Ambrose Channel. The whole cost of that dredge was charged against the Ambrose Channel improvement, but it was sold to the Philadelphia District and now, after about twenty years, it is in fair condition and still doing good work. Insurance is charged in the cost-keeping accounts for the purpose of making comparative estimates of cost, but that item is a very small one.

The U. S. Engineer Department has adopted the policy of retiring its old, out-of-date dredges. The first sea-going hopper dredges were built with wooden hulls and some of them were used for as long as thirty-five years, but they have finally all been retired from service and most of them have been sold for very small prices, practically junk. All the dredges in use at present have steel hulls, and there is no reason why they should not last longer than those with wooden hulls.

The sea-going hopper dredge does work in certain places, such, for example, as the ocean bars of many harbors, at a much less cost than such work can be done by any other type. The type of sea-going hopper dredge in use in the United States has been developed entirely by the U. S. Engineer Department. Dredges somewhat similar to the American sea-going hopper dredges are in use in Europe, but the type apparently most favored there is one which was tried in the early days of the Ambrose Channel improvement and proved not to be a success. The newer sea-going hopper dredges in use by the Engineer Department are very expensive, costing $600 000 and more. The great cost of these dredges is probably the reason why contractors do not use them, because they cannot afford to tie up so much money in a single piece of plant that may not be kept busy at all times. The extremely small cost at which these dredges can do work fully justifies the ownership and operation of a limited number of them by the Government. Their great flexibility in being able to work in crowded channels, such as those in New York Harbor, and the facility with which they may be transferred from one locality to another, also give added reasons for their use by the Government. The element of portability is shown by the fact that one of these dredges, which was built on the Delaware River, was sent to Alaska under its own power. There it worked under conditions that would have been very difficult for any other type of dredge on account of the heavy tide, the distance from base of operations, and other unfavorable conditions. The District Engineer, basing his estimates on the best data that he had been able to obtain from contractors as to prices they would probably charge for this work, considered that the use of the hopper dredge saved more than $100 000 to the Govern-

ment in the cost of this one operation alone. Considering the difficulties of doing the work with any other type of dredge, it is believed that this estimate of saving is not too high.

F. C. WARNER,* M. AM. SOC. C. E.—The Mitchell and Giesler studies, mentioned by the author, were made when the Delaware River was in its natural condition, no work of improvement having been done beyond a small amount of dredging. There is no record so far as known to the speaker that the results of these studies had been used in any way in connection with the improvement of the Delaware River until they were taken up by him in his study in 1917. The natural regimen of the river has been materially changed by dredging and the construction of dikes and jetties in connection with the projects for the 30 and 35-ft. channels, particularly in changes in currents, depths, and tidal conditions. It has been found that the mean tidal range has decreased about 0.7 ft. and the elevation of mean low water has been raised about 0.8 ft., compared with the range and elevation of the tides in the river in its natural condition. This is due largely, it is believed, to the improvement work. The existing condition of the river in 1916 rather than its natural condition, therefore, was taken as the basis for this study.

In the author's reference to the study by H. F. Flynn, M. Am. Soc. C. E., in 1918, it is noted that Mr. Flynn takes exception to this, as he believed that the natural regimen of the river should have been used instead of an artificial state. For the reasons stated, the speaker does not agree. For this study, cross-sections of the river channel were taken, in the fall of 1916, at each nautical mile, beginning at Fort Mifflin Light and extending a distance of 40 nautical miles down the river, the locations being the same as for those used in the Mitchell study. Fig. 9 shows the results of this study.

The survey could not be extended the full 46 nautical miles covered by the Mitchell study, nor could intermediate cross-sections be taken, as the field work was stopped by cold weather and ice in the river. To cover the full 46 nautical miles, cross-sections were taken from a recent survey. Examination of many charts and surveys of the river showed that at certain stretches, from 1 mile to 3 miles long, natural depths of 35 ft. or more at mean low water and a bottom width of about 800 ft. existed in both its natural and improved condition. Other stretches were found to be 35 ft. or more deep, but less than 800 ft. wide. These natural depths of 35 ft. or more that existed in the original condition of the river, and still exist at the same places in its improved condition, must be maintained according to some definite law, and it was assumed, therefore, that cross-sections could be accepted as "standard" for the volume of tidal flow at these points, from which theoretical cross-sections for natural maintenance at intermediate points could be determined. With this in view three typical, or "standard", cross-sections were selected, 3, 20, and 44.6 nautical miles from Fort Mifflin Light, respectively, the last being at the head of the Delaware Bay channel where there was a depth of 35 ft. at mean low water and a bottom width of 800 ft. For a better average cross-section at these points the arithmetical mean of three cross-sections, one above

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and one below, was used. Applying the Giesler equation, the hyperbolic curves for cross-sectional areas at mean low-water and half-tide stages were determined, from which intermediate cross-sectional areas at each nautical mile were computed.

The author's Table 4 gives the half-tide cross-sectional areas for each 5 miles from 5 to 40 nautical miles, inclusive, below Fort Mifflin Light. These curves, thus determined, platted on the low-water and half-tide profiles of cross-sectional areas obtained from the survey, are shown on Fig. 9. The low-water curve passes through five cross-sectional areas having natural depths of 35 ft. or more at mean low water, all of them being 800 ft. wide, including two "standard" cross-sectional areas used in determining the curve. They have a length of 7 nautical miles out of the total of 44.6 nautical miles under consideration. The curve also passes through, or close to, other intermediate points having a natural depth of 35 ft. at mean low water, which serves to verify the correctness of the equation. It appears safe to assume, therefore, that as this curve passed through five "standard" cross-sectional areas with an aggregate length of 7 nautical miles and the "standard" cross-sectional area at the head of the Delaware Bay channel at Nautical Mile 44.6, it should give correct theoretical cross-sectional areas for the natural maintenance of a 35-ft. channel, 800 ft. wide at all points of the remaining 37.6 nautical miles of channel of less than 35 ft. depth.

From these theoretical cross-sectional areas, widths at mean low water may be determined at each point where a reduction in the present cross-section and width is required. It is believed that this theory is entirely rational and can be applied, in connection with the location of regulation works for natural maintenance, at sections of the river channel that now require continuous maintenance dredging. The author has quoted the speaker, as to the practical application of this theoretical law.

The speaker intended to continue this study progressively by taking cross-sections annually at the same points and also study the tides, currents, river slopes, etc. With this in view automatic tide gauges were installed in the spring of 1917 at five places below Fort Mifflin Light to the head of Delaware Bay. Precise level bench-marks of the U. S. Coast and Geodetic Survey and the U. S. Engineer Office at Philadelphia, Pa., are located along the west shore of the river, to which the planes of mean low water, as determined by these gauges, may be referred. This program was interrupted, however, by his transfer to another district.

The Delaware River carries a large percentage of material in suspension, brought down from its head-waters and from its many tributaries, particularly during periods of freshets, much of which is deposited in the dredged channel. It is believed that this sediment is precipitated largely when the fresh water, with its suspended burden of silt, meets the salt water and this great mass of mud and water moves along the bottom up and down with the tides until a condition favorable to its deposit is reached, which is usually at the broad sections of the river where the currents slow down. These are the sections that require continuous dredging for maintenance. It is believed

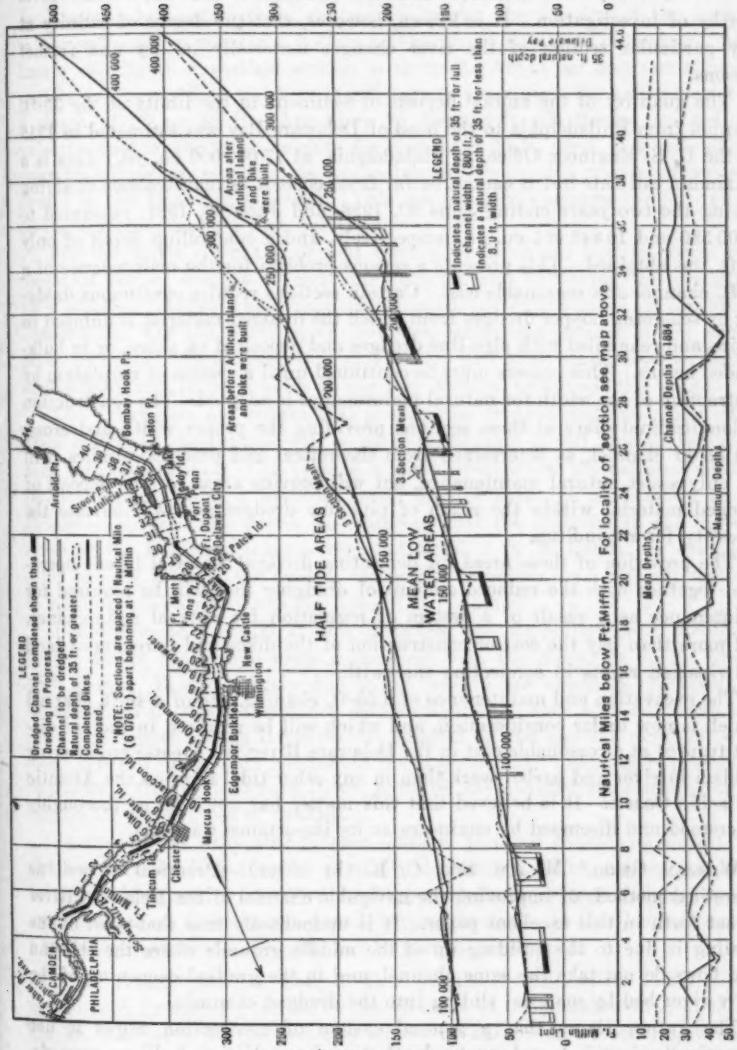

FIG. 9.—CURVES OF CROSS-SECTIONAL AREAS AND DEPTHS, 35-FOOT CHANNEL, DELAWARE RIVER.

that the degree of salinity of the water has much to do with the precipitation and deposit of sediment, but as no study of this feature has been made, so far as known, the particular sections where this occurs are uncertain. It is worthy of investigation. It is known, however, that the degree of salinity at any particular section of the river changes materially in dry and freshet seasons.

The quantity of the annual deposit of sediment in the limits of the 35-ft. channel from Philadelphia to the head of Delaware Bay was estimated in 1915 by the U. S. Engineer Office, at Philadelphia, at 17 000 000 cu. yd. This is a maximum estimate but it cannot be far from correct as maintenance dredging during the two years ending June 30, 1926, and June 30, 1927, amounted to 9 200 586 and 10 845 001 cu. yd., respectively, and a controlling depth of only 34 ft. was obtained. This presents a serious problem for the maintenance of a 35-ft. channel at a reasonable cost. Certain sections require continuous dredging by sea-going hopper dredges from which the dredged material is dumped in basins and rehandled with pipe-line dredges and deposited on shore, or in bulkheaded basins. This process must be continued until a system of regulation by contraction of the width for natural maintenance is adopted. The construction of longitudinal dikes at these sections providing the proper width and cross-section of channel, as determined from theoretical and practical studies, will not only assist natural maintenance, but will provide areas for the deposit of dredged material within the reach of pipe-line dredges and will obviate the necessity for rehandling.

The provision of these areas for depositing dredged material in one operation, together with the reduced amount of dredging that will be required for maintenance as a result of a system of regulation for natural maintenance, will more than pay the cost of construction of the dikes and purchase of land and riparian rights in connection therewith.

The excavation and maintenance of a 35-ft. channel, and of a 40-ft. channel (which is now under consideration, and which will be required in the not distant future) at a reasonable cost in the Delaware River is a greater engineering problem in river and harbor work than in any other tidal river on the Atlantic or Pacific Coasts. It is believed that this matter has not been as thoroughly understood and discussed by engineers as its importance warrants.

William Gerig,* M. Am. Soc. C. E. (by letter).—Practical as well as theoretical methods of improving the navigable channel of the Delaware River are set forth in this excellent paper. It is undoubtedly true that most of the shoaling is due to the building up of the middle grounds where the ebb and flood flows do not take the same channel, and to the gradual deepening of the entire river bed by material sliding into the dredged channel.

The author shows that a general system of contraction works is not economically justified and contends that a few additional dikes, properly placed for guiding the ebb and flow into one channel, will considerably reduce the maintenance. The writer believes that such dikes, although expensive to

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build, are justified, and can be placed so as to do a minimum amount of damage to adjoining property.

The material coming into the channel from the sides must be removed by dredging because, without contraction works, the velocity of the current is not sufficient to scour the bottom at this great depth. The average maximum velocity in a vertical section is about 2.4 knots per hour, or approximately 4.0 ft. per sec., which is insufficient to scour even the soft material at a depth of 30 or 35 ft. According to Kennedy's silt theory, the critical velocity, that is, the mean velocity for a channel 35 ft. deep, just sufficient to prevent silting or scouring, is approximately twice the maximum velocity found in the Delaware Channel. This low velocity accounts for the failure of the agitation method that was tried.

For the upper river, the present method of dredging, that is, using self-propelled hopper dredges to clear the channel, and delivering the dredgings to a pipe-line dredge where they are pumped out of the river, is considered economical. On the lower river, however, where the haul does not exceed 30 to 50 miles, it is believed that it will be found more economical to dispose of the dredgings in deep water in the Bay Section, or outside the Capes.

The writer does not believe that placing the dredged material along either or both sides of the channel will be effective, because it will immediately come back into the channel. This is done in some rivers where the material encountered is usually sand, a material much heavier than the mud found in the Delaware River. The writer has observed that the material in the disposal areas of the Delaware takes a very flat slope, 1 ft. in 100 ft., or even less, after passing through the dredge. It would, therefore, be a difficult material to retain on the already moving sides of the channel.

For delivering material at long distances, the sea-going hopper dredge of proper design has proved more economical than the dipper or bucket dredge with scows. This is largely due to its facility for handling, its speed, its capacity and its non-interference with navigation. Also, each cut made by a sea-going hopper dredge improves the channel by the amount removed, while other types of dredges do not benefit the channel until dredging is practically completed. On the Delaware River, however, the channel deteriorates rapidly behind the latter types of dredges.

In economical work with hopper dredges, it is, of course, important that the economic loading time be carefully determined. The method for ascertaining this is well understood by hopper-dredge operators and has become standardized on Government-owned hopper dredges. If this loading time is not carefully obtained, the cost may be greatly increased.

F. C. Boggs,* M. Am. Soc. C. E. (by letter).—The writer is heartily in accord with Professor Spofford in his views as to the possible value of models for an indication of the probable effects of harbor and river improvement works. He trusts that one or more laboratories will be established in the near future, where these and other problems of hydraulics may be studied. In this connection attention is invited to the recommendations along these lines,

* Col., Corps of Engrs., U. S. Army; Dist. Engr., Philadelphia, Pa.

made by Maj. Gen. Edgar Jadwin, Chief of Engineers, U. S. Army, M. Am. Soc. C. E., in his report on flood protection works for the Mississippi River.

Professor Spofford and Mr. Sherman raised the question as to cost-keeping methods. General Taylor has quite fully covered this phase of the subject. Professor Spofford also pertinently suggests that if costs for work done by the Government plant are so low, it might be advisable to use this plant in Philadelphia Harbor instead of having dredging work there done by contract. The self-propelled hopper dredge, used by the Government on the Delaware River, would not be satisfactory for work in the restricted limits of the harbor. The numerous passenger and car ferries, small boats, and towed barges, together with larger boats at anchor or entering or leaving their slips, would make the operation of a self-propelled dredge unsatisfactory and dangerous. The width of the harbor, about 1 800 ft., does not offer the opportunity of alternate routes as does the much wider Hudson River at New York. Moreover, the many under-water cable lines which can be readily handled by clam-shell or dipper dredges, would be subject to serious damage from the suction drag-head of a self-propelled dredge. While these difficulties are not insurmountable, they would add materially to the cost, which would be still further increased on account of the long run necessary to reach a disposal area. These dredges, when loaded, draw from 18 to 20 ft.

Mr. Sherman comments on the statements made by the writer in regard to the municipally owned piers. In justice to those who have control of the policy of handling these piers, it should be stated that the Pennsylvania Courts have decided that, except by Special Act of Legislature, private piers as well as piers leased from the City of Philadelphia, must be open to all comers. As one jurist expressed it, "the owner of a wharf is under the same obligation as an inn-keeper to receive those who call, provided he has room, and they tender a reasonable compensation." Despite the legal aspects, the writer is still of the opinion that one or more municipally owned and operated piers would be advantageous.

Professor Spofford asks why inland ports, such as Philadelphia, should be developed by the Federal Government. This presents a matter which cannot be fully covered in a brief paper. The writer believes that a consideration of the comparative costs of transportation by rail from Boston, New York, or Baltimore, to the 5 000 000 or more people in the Philadelphia area, with costs of water deliveries through the Port of Philadelphia, will give a convincing answer to this inquiry.

Several of the discussors mention the increased height of the mean low-water plane in recent years, in the river above Artificial Island. The writer agrees that this is undoubtedly due to the constriction of the river between Artificial Island and Reedy Island Dike. Whether this constriction, with the resultant raising of the low-water plane, has been harmful, is open to question.

The writer cannot agree with Mr. Warner in the theory that all the maintenance dredging required is due to silt originating in the upper river and its tributaries and deposited from suspension in the lower river. While undoubtedly some of the material removed in maintenance work is of this

origin, the writer believes that the main cause of the formation of shoals in the channel is the gradual movement of the soft material forming the river bed. Under natural conditions he believes that the bed of the stream at any cross-section would maintain, with little change, a semi-permanent profile. If this is modified by dredging a channel or by placing over-burden on any part of the profile, the equilibrium of forces is disturbed. Cross-currents produced by the ebb and flood tides, which it may be noted attain approximately the same maximum strengths, facilitate this movement down the sloping bed of the river into the channel.

The writer again desires to state that he is not opposed to dikes constructed for the purpose of regulating flow. He is opposed to dikes in the tidal section of the Delaware River, constructed for the purpose of inducing strong currents, since such structures, due to practical equality of ebb and flood currents, would tend to induce scour during both these stages of the tide. The result would be that about the same amount of material would be carried up stream on the flood as would be carried down stream on the ebb. In his opinion, dikes to eliminate undue widths of the stream, or to guide currents in lines parallel to the channel, would be advantageous. Such dikes should be constructed only after careful and extensive current studies at each locality where shoaling occurs.

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Paper No. 1703

THE HEAD-WORKS OF THE IMPERIAL CANAL*

By C. E. Grunsky,† Past-President, Am. Soc. C. E.

With Discussion by Messrs. E. S. Lindley and C. E. Grunsky

Synopsis

The diversion of water from the Colorado River for the irrigation of lands in the Imperial Valley, California, has been fully described in papers previously submitted to the Society.‡

In 1916, mainly on account of difficulty with the silt carried by the Imperial Canal, it became necessary to make considerable improvements which included an up-stream extension of the canal for about 1 mile and the building of an intake structure which is known as the Rockwood Gate. This gate was designed for erection on a sand foundation. It is a structure with seventy-five openings between piers and extends for more than 600 ft. along the bank of the river. For about 400 ft. of this distance, the foundation is fine sand which has been prevented from being piped out from under the gate by placing upon it, without pile support, the floor of the structure, heavily weighted by a box of sand carried by the piers which sub-divide the gate into sluice-like openings. There has been no trouble with settling since its completion.

The Imperial Canal and Its Intake Structure

The Imperial Canal, so called because it was built for the irrigation of lands in that part of the Salton or Coahuila Basin now known as Imperial Valley, is also known as the Alamo Canal. Its head is in California on the right bank of the Colorado River just up stream from the International

* Published in November, 1927, *Proceedings*.

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‡ "The Lower Colorado River and the Salton Basin", by C. E. Grunsky, Past-President, Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LIX (1907), p. 1; also, "Irrigation and River Control in the Colorado River Delta," by H. T. Cory, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXVI (1913), p. 1204.

Boundary line between California and Mexico. Its course is first southerly into Mexico, thence westerly and northwesterly back into California. About 40 miles of the main canal are in Mexico. This canal supplies water to about 450 000 acres of cultivated lands in California and to about 200 000 acres in Mexico. Not an acre of the cultivated land could produce crops without irrigation, normal rainfall being only 2 to 3 in. per year.

Within a few hundred yards of the International Boundary line, at the base of Pilot Knob which rises toward the west, is a regulating gate that was built in 1905 on good rock foundation. This was originally provided with eleven openings or sluices through which the flow was controlled by Taintor gates. Drift and silt were troublesome and interfered seriously with the manipulation of the gates, wherefore, in the course of time, several of the original gates were replaced with Stoney gates. There was an open cut, a little more than ¼ mile long, from this gate to the river, in which a dredge had to be maintained to stir up silt deposits and prevent reduction of capacity, which were always likely to occur there on a falling river. The importance of maintaining capacity will be appreciated when the fact is taken into account that about 2 000 sec-ft. of water in winter and more than 5 000 sec-ft., at the height of the irrigation season (in 1927 already in excess of 7 000 ft.) had to be sent down the canal, practically without interruption.

The water entering the canal carried from about 0.3% to 4.0%, averaging probably about 0.8%, of very fine silicious silt in suspension and, besides, had an appreciable bed or bottom load of fine sand. At the time the modification of this intake arrangement was recognized as essential, all the silt which entered the canal was carried by the water down the main canal, into the laterals, into the small irrigation ditches, and, in large part, out on the fields, lodging in the canals, ditches, and on fields, frequently at points where its accumulation became increasingly embarrassing. The quantity of this silt transported down the main canal, estimated at a weight of 100 lb. per cu. ft., was computed, from the data available, to be about 12 000 acre-ft. per year. It has been reported by M. J. Dowd, Assoc. M. Am. Soc. C. E., Chief Engineer of the Imperial Irrigation District, to have reached a peak of 24 900 acre-ft. in 1923 and to have averaged more than 14 000 acre-ft. during the period, 1915-1927.

There was a twofold purpose in the up-stream extension of the canal and the building of a gate in the bank of the river:* First, the structure was to be of such a type that it would reduce the incoming silt to a minimum, mainly by holding back the bottom load of the water; and, second, in the mile stretch of canal between the new intake gate and the original regulating gate, there would be ample space in which to operate dredges with which it was proposed to maintain extra depth, thereby providing opportunity for the settling of some of the heaviest of the silt in suspension.

The quantity of solid material in suspension in the water of the Colorado River, at Yuma, Ariz., 8 miles up stream from the head of the Imperial Canal, had been determined by the U. S. Reclamation Service which commenced

* The project was recommended, designs were approved, and general supervision was exercised by the late George G. Anderson, M. Am. Soc. C. E., and the writer, Consulting Engineers to Imperial Irrigation District. The preliminary design was prepared by the writer.

its sampling in 1909. The records thus obtained established the fact that there is somewhat less material in suspension at the water surface than at mid-depth, or near the bottom of the river. These silt determinations indicate that there is about 12% more silt in the water of the Colorado near the bottom than at the surface. It appeared, therefore, worth while to arrange the intake so that mainly surface water would enter the canal. Consequently, the gates of the Rockwood Intake were planned to be of the over-fall type, thereby not only checking the movement of silt on the bottom in the head of the canal, but also, in some measure, reducing the average quantity of silt in suspension.

Two dredges were provided for the inter-gate section of the canal and, for a time, in the period from 1918 to 1920, these removed upward of 500 000 cu. yd. per month of fine sand from this section without making any permanent impression on the depth. These facts relating to the magnitude of the silt problem are noted merely to emphasize the urgent need which had long been recognized for the Rockwood Gate. It is not proposed, however, to enter on a full discussion of the canal's silt problem at this time.

The preliminary design of the new intake structure which is built into and replaces more than 600 ft. of river bank, was made on the assumption that it would have to be constructed on a foundation of fine sand. As built, its location is at the base of a low gravel-topped hill somewhat farther up stream than had at first been planned, with the resulting advantage of securing for its up-stream one-third, or for about 200 ft., a foundation of slightly cemented sand, a material which, although quite soft, can be cut to, and will stand with, a vertical face. A similar formation, exposed at a number of places in the hills along the river at and below Yuma, is soft enough to be carved readily with a knife. Except for the large excavation required, the gate would have been shifted entirely into this cemented sand formation.

The floor of the structure is at two levels. The up-stream gates, 27 in all, have their sills 4 ft. lower than the remaining 48 down-stream gates. The gates with the lower sills are on the better foundation. The structure consists in the main of a heavy concrete floor with cut-off walls at the up-stream and down-stream edges (Fig. 1). Piers rise from the floor and are spaced so as to leave openings about 6½ ft. in the clear between their sides. The piers carry a sand-box, 22 ft. wide by 10 ft. deep, which serves the purpose of loading the floor. The weight of the fill in the sand-box, transferred by the piers to the floor, is relied on to hold in place the sand on which the structure rests.

At the up-stream edge of the structure, sheet-piling has been driven. Its purpose was not only to reduce the movement of water through the underlying sand, but also to serve as a form for the concrete cut-off walls. It will be noted that this piling gives no support to the structure, being placed in front of, and not under, the concrete so that it does not interfere with the transmission of the full weight of the intake to its sand support. It was proposed to drive this sheeting to a depth of 30 ft. below the floor level. A jet was used in driving the sheet-piles which were of the built-up Wakefield type, but so much difficulty was experienced that, in the finished work, few are more than 20 to 25 ft. in the sand. The proposition to drive them to the greater

depth originated with C. R. Rockwood, Chief Engineer of the District, and his assistants, who based their conclusion as to the possibility of doing this on long experience with similar work, although of less magnitude, in the Lower Colorado River region. In the very fine sand, at the site of the intake, the piles would drive readily about 10 or 12 ft. It was difficult to control them

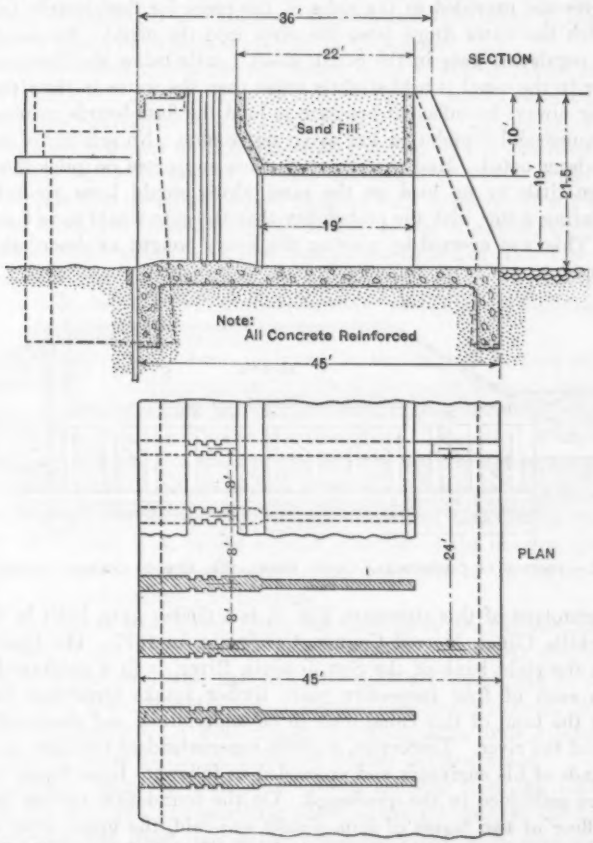

FIG. 1.—PLAN AND SECTION, ROCKWOOD GATE AT HEAD OF THE IMPERIAL CANAL, CALIFORNIA, BUILT 1918.

during further driving. They could not always be held snug against the preceding pile and despite the liberal use of water from a jet, soon required such hard blows that many of them went to pieces and had to be pulled. The sand in which this difficulty was experienced is very fine and has little, if any, admixture of clay. It appears very compact and firm under water, but partakes of the nature of quicksand when jarred or caused to vibrate.

Along the down-stream edge of the structure, at both abutments and across the canal, is a heavy rock-fill. Except for a small portion this was, according

to the original program, to be placed from time to time after the gate went into use, all settlement of the fill to be made good as it developed.

Up stream from the sand-box the piers rise several feet above the extreme high-water stage of the river and are united at the river face of the structure by a slightly ornamented cornice which gives a pleasing aspect.

Grooves are provided in the sides of the piers for flash-boards (stop-logs) over which the water drops from the river into the canal. By means of the original regulating gate in the canal, about 1 mile below the Rockwood Gate, the water in the canal is held slightly lower than the water in the river so that there may always be sufficient pressure to hold the flash-boards in place.

The quicksand foundation has given no trouble. No settling of the structure has been noted. Had the structure been supported on piles there would have been little or no load on the sand which would have yielded readily to percolating water with the probability that the gate would have been undermined. This was averted by placing the heavy weight as described directly on the sand.

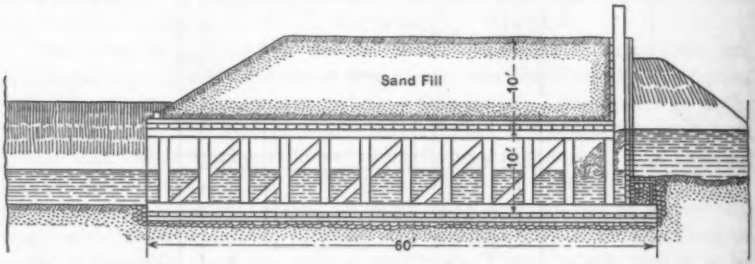

FIG. 2.—SECTION OF CHOWCHILLA CANAL REGULATOR, MERCED COUNTY, CALIFORNIA, BUILT 1877.

The prototype of this structure, Fig. 2, is a timber gate, built in the head of Chowchilla Canal, Merced County, California, in 1877. The head of this canal, on the right bank of the San Joaquin River, is in a quicksand formation. In each of four successive years timber intake structures had been placed in the head of this canal only to be undermined and destroyed by the next rise of the river. Thereupon, a ranch superintendent took the matter out of the hands of his engineers and proceeded as follows: Loose blocks of sandstone were embedded in the quicksand. On the foundation surface thus prepared, a floor of two layers of 2-in. planks was laid, the upper layer at right angles to the lower one. This floor was about 6 ft. below the bed of the river. On this floor a framework of 10 by 10-in. timber was placed which, in turn, supported a 10-ft. deep box, 10 ft. above the floor, filled with sand. Along the up-stream margin of the floor a box-like structure, 5 ft. high, filled with rock was provided, thereby creating a drop of 5 ft. from river to gate floor. The regulating gates moved between timbers which rose from the down-stream edge of this box and gave support to the head-wall of the main sand-box. The over-all length of this structure was 60 ft. The width between its side walls was 23 ft. When last seen by the writer, after it had been in service more than 20 years, it was still in good condition.

DISCUSSION

E. S. LINDLEY,* M. AM. SOC. C. E. (by letter).—The account of the work given in this paper shows practice that differs in several respects from that followed in India. It seems worth while to describe the latter for comparison; but there is no intention of implying, without any knowledge of local conditions, that it would have been applicable in California.

The writer wishes to discuss two points namely, silt exclusion and building on a foundation of sand. The 500 000 cu. yd. per month reported removed by dredges are equivalent to 3 620 acre-ft. per year. Comparing that figure with the earlier estimate of 12 000 acre-ft. per year of silt admitted, and the later estimate of 14 000 acre-ft., with a maximum of 24 900 acre-ft. in one year, it is clear that dredging cannot compete with measures of silt exclusion. Even if as much silt were dredged in a year as was admitted, trouble would still be experienced at times. Under Panjab conditions of river flow, probably about three-quarters of the total silt of the year passes in the four worst months (a flood season when Himalayan snows melt and are supplemented by monsoon rainfall).

The new Rockwood Gate was designed "so that mainly surface water would enter the canal". The design seems to have followed the common tacit assumption that provision of a high sill excludes from the regulator any water which flows in the parent channel lower than the plane of the sill. If the matter is regarded from the point of view of the draw or suck exercised by the offtake at different points in the cross-section of the parent channel, it will be realized that the cleavage between the water taken in and that passed on, is vertical and not horizontal; and that, except for the behavior of the heaviest particles in a vertical current, a high sill admits the same silt as an under-shot gate, because it does not change this cleavage.

Investigation is now showing that the cleavage is sensibly vertical over the greater part of the depth in the parent channel; but near the bed, where forward velocities are lowest and silt conditions are worst, flow to the offtake comes from a greater width, to a degree considerably more than would ordinarily be expected. A side offtake, therefore, must draw a large share of the more silty water to the exclusion of cleaner surface water.

The author refers to local observations showing that the bottom water contains the most silt. If sampling could be carried close to the bed without disturbing the bed itself, the water would be found to be even dirtier than stated.

Observations at other places have shown, not only that the average silt charge is greater at the bottom than near the surface, but that this concentration is greater in the coarser grades of the silt carried. Heavy silt is absent from the surface water, and the heaviest is confined to the bottom water, unless the current is turbulent. Viewed in the light of these considerations, it can be understood that a side offtake is a selector of silt, or the opposite of a

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filter. In developing the design of offtakes intended for silt exclusion from this point of view, the object is to secure the utmost concentration of silt in parts of the approaching current which are then excluded. The first step involves conducting the flow of the parent channel for a sufficient distance in a smooth channel, at a velocity that just avoids depositing silt on the bed, and also avoids eddies and boils as much as possible.

For the second step, a device that is applicable in some cases, consists of smoothly curved vanes on the bed, which guide the dirty water away from the offtake, give the whole current a twist, and thus also bring it clean water on the surface. This device first suggested by Mr. H. W. King, of the Panjab Irrigation Department,* is thus the opposite to that used by Benjamin F. Groat, M. Am. Soc. C. E., to exclude ice.†

Another suggestion, applicable on a large scale, is due to the late Mr. F. V. Elsden, of the Panjab Irrigation Department. For a new work he would build a regulator in two stories across the parent channel, in the position of the under-sluices of a Panjab head-works. The gates of the lower story would escape dirty bottom water down the river, while those of the upper story would take the cleaner upper flow to be conducted away in the canal.‡

A plain high and long sill also has some effect in excluding the coarsest silt, without affecting the plane of cleavage of the parent channel. Any given grade of silt could approximately be excluded if the flow had nowhere an upward vertical velocity exceeding its rate of settlement in still water. This presupposes prevention of eddies to stir up the bottom silt or cause local upward currents, and a slight current along the face of the sill to prevent the excluded silt from banking up there until it all enters again.

Referring to the second point for discussion, when a work is built on a pervious foundation to head up water, and without a cut-off to an impervious stratum, the difference of head causes flow through all the pervious medium, and there is a regular drop of head along any stream line. *Prima facie,* it might be expected that the easiest path (along which the greatest flow occurs) would be the shortest; that is, the envelope embracing the work and cutting off any re-entrant angles. However, observation of static pressures under such works has shown that distances in which equal falls of pressure occur should be measured along the "line of creep", or junction between the work and foundation, along both vertical and horizontal lines. The weight of the work must exceed the total upward pressures indicated by the line of hydraulic gradient thus drawn. According as to whether or not the work is tied together by reinforcement, the weight must be suitably distributed for stability, or must be sufficient from point to point.

The shorter the line of creep the steeper the gradient and the greater the velocity of flow. For any given material there is a limiting velocity which must not be exceeded to avoid sand being blown at the toe of the work, with consequent piping. In that material such a velocity will be caused by a certain gradient, which is the (maximum) limiting gradient for that material. There-

* Panjab Eng. Congress, 1918 and 1920.

† *Transactions,* Am. Soc. C. E., Vol. LXXXII (1918), p. 1138.

‡ Panjab Irrig. Branch, *Paper No. 25.*

fore, with a given heading up, this gradient gives the minimum length of line of creep for the work. To exceed that length would add unnecessarily to the dimensions of the work and the weight required to balance upward pressures.

The limiting gradient for the sandy beds of Panjab rivers about 50 miles from their emergence from the foot-hills is generally taken as 1 in 15. In the finer loam of the plains between rivers, a gradient of 1 in 4 is used, but for finer pure sand it is flatter than 1 in 15. The length of the line of creep under the Rockwood Gate is about 80 ft. With the canal empty, the maximum head would be nearly 11½ ft., which would give a gradient of 1 in 7. The gradient for the Chowchilla regulator would seem to be about 1 in 6. As the sand is described as very fine and free of clay, these works seem very short, on these data and judged by this criterion. With a given length of line of creep, if the gates are moved down stream a greater part of the work has water at full depth standing on it, leaving a smaller part of the upward pressures to be balanced by the weight of the work. Therefore, it is economical to provide an impervious up-stream apron, especially as it is generally desirable to protect the bed against scour, and to have a smooth floor, thereby causing less eddying and less raising of silt in the stream.

C. E. GRUNSKY,* PAST-PRESIDENT, AM. SOC. C. E. (by letter).—Several points of importance in connection with the planning of offtake structures from silt-laden streams are brought out by Mr. Lindley. He states that it is not only water from the top of the stream that will go out over the top of flash-board gates, but water from all depths. In this he is quite right, but when the over-falling sheet of water is generally less than a foot in depth and the approaching water, or rather the water flowing along the face of the offtake structure, has various depths up to 20 ft. or more, it is only reasonable to assume that some advantage will result from the over-fall system. Furthermore, the sand or silt moving by saltation close to the bottom, which should be classed as bed-load, and the sand which actually rolls along the bottom, are excluded by the system.

As stated by Mr. Lindley, there is, undoubtedly, some flow of water through the sands under the Rockwood Gate, but the gradient from the water surface in the river to that in the canal is not as steep as he has assumed. There is a regulator in the canal about 1½ miles below the Rockwood Gate and the drop from the river to the canal is controlled by this regulator. There is no definite information at hand as to the allowable maximum, but ordinarily the drop is only a fraction of 11½ ft., which is the amount introduced into the calculation by Mr. Lindley. This value (11½ ft.) should be regarded as a possibility only when the canal is out of commission during fairly high water —a condition which it is thought will never obtain.

* Cons. Engr. (C. E. Grunsky Co.), San Francisco, Calif.

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Paper No. 1704

THE SCIENCE OF FOUNDATIONS—ITS PRESENT AND FUTURE*

By Charles Terzaghi,† M. Am. Soc. C. E.

With Discussion by Messrs. Charles R. Gow, E. P. Goodrich, Victor A. Endersby, George Paaswell, C. C. Williams, Herbert Chatley, J. Albert Holmes, Jacob Feld, W. S. Housel, Frank S. Bailey, T. L. Condron, Lazarus White, R. D. N. Simham, Frank A. Marston, Arthur M. Shaw, D. P. Krynine, S. P. Wing, M. L. Enger, Albert Jorgensen, and Charles Terzaghi.

Synopsis

The paper reviews the present state of the science of foundations, its principal shortcomings, and the possibilities for its improvement.

The principal shortcomings were found to be: First, the practice of selecting the admissible soil pressure regardless of the area covered by the individual foundations and irrespective of the maximum differential settlement that the superstructure can stand without injury; second, the practice of computing the bearing capacity of the piles by the *Engineering News* formula regardless of the character of the soil; and, third, the practice of considering the bearing power of the individual piles as a sufficient guaranty that the bearing capacity of the entire foundation will be adequate.

In his discussion of these topics the writer tries to explain the reasons for the inconsistencies that are often experienced in attempts to interpret the results of loading tests for designing purposes. Concerning the pile-driving formulas the writer presents physical arguments why, for certain soils, no pile-driving formula can possibly furnish reliable information concerning the bearing capacity of the tested pile. For those soils for which the

* Presented at the meeting of the Structural Division, New York, N. Y., January 20, 1927.

† Prof., Foundation Eng., Mass. Inst. Tech. Cambridge, Mass.

pile-driving formulas can be used to advantage, the *Engineering News* formula is found to furnish values which are by far too small, provided a drop-hammer is used and the penetration per blow amounts to less than ½ in.

Progress in the field of foundation engineering is handicapped essentially by the absence of reliable information concerning previous construction experience. Again, the lack of reliable information is due to inadequate description of the soils and to an interpretation of the observed facts which, in many cases, is inconsistent with the laws of physics and mechanics. The first step toward improving conditions should consist in establishing an adequate soil classification.

Thus far, related efforts have failed, because they represented attempts to classify the soils according to properties that have no bearing at all, or no well-defined bearing, on the behavior of the soil under load.

The writer presents a list of those soil properties that determine the character, the amount, and the speed of the settlements. He shows how the knowledge of these properties can be utilized for establishing a soil classification suited for its purpose and briefly discusses the difficulties associated with such an attempt.

The problems of foundation engineering are as inseparably associated with the problems of structural engineering, as is the shadow with the light, because one cannot conceive of a structure without a foundation. Therefore, they are among the most vital.

Effect of Type of Building on Admissible Settlement

Suppose it is desired to construct a reinforced concrete building on a lot selected and bought by the future owner. After inspecting the ground in the light of previous experience, it is decided to dispense with piles and put the building on individual spread footings. The mental operation leading to this decision is experienced almost daily, and is familiar to every engineer. This apparently simple decision may be analyzed by common sense. By spreading the weight of the building over the base of sufficiently large footings, the pressure can be reduced to a certain value, q, per unit of area. Under the influence of the pressure, q, the footings will settle through a distance, S. Perfectly uniform settlement will not hurt the building. However, because of the variations in the distribution of the live load, the pressure may vary between q and $0.8\ q$ (to name an arbitrary figure) which corresponds to a difference in settlement between S and $0.8\ S$. In addition, it may be that the compressibility of the ground varies to such an extent, that the settlement produced by a load, q, may range between S and $0.5\ S$ (which, again, is an arbitrarily selected figure, the actual value depending on circumstances). Hence, a difference, amounting to as much as $S - 0.5 \times 0.8\ S = 0.6\ S = H$, may be expected between the settlement of the individual footings. In the formula,

$$H = S - 0.5 \times 0.8\ S = 0.6\ S = n\ S \ldots\ldots\ldots\ldots\ldots (1)$$

the factor, n, represents the numerical effect of both the variations in live load and the variations in the compressibility of the soil. The value, H, will rule

all further considerations. If the building is statically determinate a value of several inches for H would be permissible. On the other hand, if it is statically indeterminate, the allowable intensity of the secondary stresses must be selected first. Consider, for example, that an increase of 15% in certain maximum bending moments would be admissible. Then, the maximum value, H, for the difference in settlements is at once determined. It may range between 0.1 in. and 1.0 or 2.0 in., according to the character of the building, the distance between the columns, and the cross-section of the beams. Hence, the decision to admit an increase of 15% in the dangerous stresses, would require selecting the soil pressure, q, anywhere between limits as far apart as 1 and 10 or 20, according to the character of the building. Selecting the value, q, on the other hand, irrespective of the structural characteristics of the building, would mean either ultra-conservative wastefulness or danger, according to the circumstances. Since this is done in most of the cases dealt with in actual practice, one cannot help feeling the necessity of a thorough reform. Every foundation design ought to be started by roughly computing the maximum admissible value of H.

Relation Between Settlement, Size of Loaded Area, and Depth of Foundation

The second inconsistency that confronts the foundation engineer daily, concerns the relation between the settlement, S, and the diameter of the loaded area. In selecting the admissible unit soil pressure, should the width and the length of the loaded area be taken into consideration? Consider, as an example, that some one makes a bending test on a 10-in. by 10-ft. I-beam, freely supported at both ends, and finds that the beam breaks under the influence of a concentrated load of P-tons. He publishes the results of his test, and as a consequence, in future practice, the designers would consider a load of $\frac{1}{4}$ P as a safe load for the beam, irrespective of the span and mode of support. In the light of present knowledge, such procedure would be considered simply absurd. However, in the field of foundation engineering it corresponds precisely to what is actually done. The safety of the building depends on H ($= 0.6\ S$) not exceeding a definite value. The value of S depends as essentially on the diameter of the loaded area and on the depth of the foundation as the load causing the rupture of the I-beam depends on the span. Nevertheless, all tables of admissible soil pressures, given in textbooks and building codes, contain the values without any indication as to what areas and to what depths of foundation they apply. They can well be compared with tables containing the loads under which I-beams of different cross-sections broke down, without mentioning the distance between supports.

The rather indifferent attitude of practising foundation engineers toward the important influence of these two items—the dimensions of the loaded area and the depth of foundation—is essentially due to the apparent inconsistencies of related experiences which seem to contradict each other, to such an extent that an attempt to digest them appears to be hopeless. However, by analyzing them and correlating them with present theoretical knowledge, one finds that they already represent a fund of most useful information, and

that the apparent contradictions do not exist. The following facts may serve to illustrate these statements.

In 1916, the Foundation Committee of the Austrian Society of Engineers and Architects, in Vienna, made a series of loading tests for the purpose of investigating the influence of the shape and size of the loaded area on the amount of settlement in a typical cohesive Vienna soil (a variety of loess).* The tests were performed on the bottom of a trench, 10 ft. deep and 5.75 ft. wide, protected by a roof. The loaded soil consisted, according to a statement issued by the investigators, of 27% of sand with grains larger than 0.2 mm., 31% of sand with grains less than 0.2 mm., and 42% of very fine-grained constituents. The water content was said to range between 11.2 and 13.2 per cent. When moulded into cubes and dried, the soil developed a cube strength of about 35 tons per sq. ft. The tests included load settlement, time settlement, and rebound observations on square and on round plates with areas of 0.053, 0.110, 0.672, 2.700, and 6.100 sq. ft. Fig. 1 shows some of the results obtained. They include the settlements of plates with different diameters under loads of 2 and of 4 tons per sq. ft., and the rebound of the soil

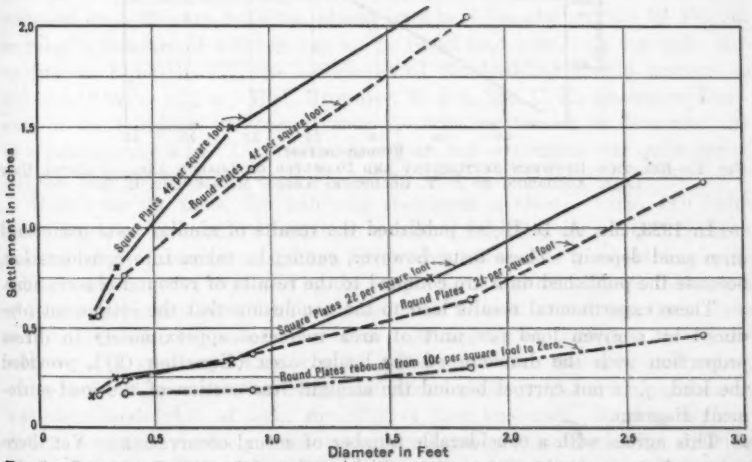

FIG. 1.—RELATION BETWEEN SETTLEMENT AND DIAMETER OF LOADED AREA FOR VIENNA LOESS AT EQUAL UNIT LOAD ACCORDING TO DR. FRITZ EMPERGER.

produced by removing a load of 10 tons per sq. ft. (Computed from the rebound, corresponding to the removal of smaller loads.) Fig. 1 plainly demonstrates that the settlement produced by a given load per unit of area increases almost in direct proportion with the diameter of the loaded area, or, in algebraic terms,

$$S = c \times d \times q \text{ (more or less)} \ldots\ldots\ldots\ldots (2)$$

* According to a manuscript, "Der Wiener Löss und seine zulässige Belastung," by Dr. Fritz Emperger, received in May, 1926. According to Dr. Emperger's letter of March 16, 1926, an extract of this paper has been published in *Die Bautechnik*, Berlin, *Le Genie Civil*, Paris, and *Gewappend Beton*, Amsterdam.

wherein, c is a constant of the soil. The shape of the upper set of curves is obviously due to the fact that the load-settlement curve for each one of the individual plates is, in itself, more or less parabolic.

A few years ago, A. T. Goldbeck, Assoc. M. Am. Soc. C. E., carried on some research work for the U. S. Bureau of Public Roads, on artificial mixtures of sand and clay for the purpose of investigating the relation between the area of a bearing block and the settlement produced by a given unit load.* The area of the bearing blocks ranged from a few square inches to 9 sq. ft. The results of the tests made on plastic mixtures were of the type shown in Fig. 2. They indicate, as do the Austrian tests, that for cohesive soils the penetration produced by a given unit load increases in direct proportion with the diameter of the loaded area.

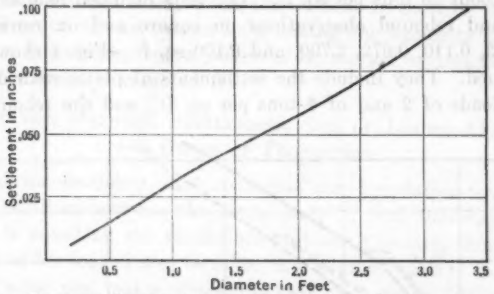

FIG. 2.—RELATION BETWEEN SETTLEMENT AND DIAMETER OF LOADED AREA AT EQUAL UNIT LOAD, ACCORDING TO A. T. GOLDBECK, ASSOC. M. AM. SOC. C. E.

In 1923, Mr. A. B. Bijls† published the results of similar tests performed on a sand deposit. These tests, however, cannot be taken into consideration, because the published data are confined to the results of rebound observations.

These experimental results lead to the conclusion that the settlement produced by a given load per unit of area increases approximately in direct proportion with the diameter of the loaded area (Equation (2)), provided the load, q, is not carried beyond the straight line section of the load-settlement diagram.

This agrees with a considerable number of actual observations. Yet there seem to be quite striking exceptions to this rule. A loading test was made by the Chicago Union Terminal Company, in Chicago, Ill., on two full-sized caissons, resting on the surface of a layer of hardpan, approximately 60 ft. below the surface of the ground.‡ The diameters of the bearing area were 4.3 ft. and 8.5 ft., respectively, and the settlements produced by a load of 10 tons per sq. ft. amounted to $\frac{1}{8}$ in. and $\frac{5}{8}$ in., respectively. Hence, although the larger diameter was equal to only twice the smaller, the settlement of the larger pier was five times greater than that of the smaller one. According to Equa-

* "Researches on the Structural Design of Highways by the United States Bureau of Public Roads," *Transactions*, Am. Soc. C. E., Vol. 88 (1925), p. 264.

† *Le Génie Civil*, May 26, 1923, p. 490. Abstract of a report by M. Wolterbeck to the Ministère des Travaux Publics des Pays Bas, 1922.

‡ *Journal*, Western Soc. of Engrs., February, 1924.

tion (2), it should not have been more than twice as large. This fact seems to indicate that the formula represents an optimistic statement.

On the other hand, experience shows that, under certain circumstances, the settlement of the larger area may be very much smaller than the value obtained by the formula. Numerous loading tests made on the plane surface of a natural bed of well-compacted sand, have shown that the settlement of a single bearing plate, with an area of 1 sq. ft., loaded with 3 tons, is never less than about ⅛ in. Hence, according to Equation (2), a foundation 20 ft. square, resting on a firm sand deposit, should settle 2½ in. under a load of 3 tons per sq. ft. Every engineer who has had experience with foundations on firm sand can testify that the settlement will be very much less, certainly not exceeding ½ in. In 1922, the Society's Special Committee on Bearing Value of Soils for Foundations, etc., published the results of loading tests performed on a building lot in San Francisco, Calif.* A stratum of soft, clean, or sticky sand, with occasional layers of sand clay and yellow clay was encountered. It was tested at eight different points. Under a load of 4 800 lb. per sq. ft., the settlement of the bearing plate ranged between 0.04 and 0.17 in., averaging 0.10 in. For the past few years the lot has been occupied by a 22-story building, placed on a raft foundation, 218 by 152 ft., exerting a pressure of 4 800 lb. per sq. ft. (dead load only)† on the soil. According to Equation (2), the settlement of the building should amount to 152×0.10 in. = 15.2 in. H. J. Brunnier, M. Am. Soc. C. E., Designing Engineer of the building, has found‡ that the total settlement to date amounts to approximately 2 in. During the first year, the settlement was quite rapid, but the rate has been less each succeeding year.

Reviewing the facts, the following statement is clear: First, two independent sets of experiments have been quoted concerning the relation between the size of the loaded area and the settlement. Both sets have led to practically the same result, expressed by the simple Equation (2). Then, the results of these experiments have been compared with those of two observations made on a large scale. In one of these cases (Chicago Union Terminal tests), the settlement of the larger area was 2.5 times more than the computed value; while in the other case (San Francisco office building), the actual settlement was about one-eighth of what should have been expected. Suppose a foundation company sincerely tries to obtain some reliable basis for drawing conclusions from the results of its loading tests. The engineers of the firm make their observations year by year, accumulate the data in their files, and, finally, when they try to correlate the data, they face grotesque contradictions of the type quoted. They cannot be blamed if they give it up in disgust, loading tests, observations, and all. However, if they would examine the data in the light of applied mechanics, they would first realize the following facts, which the writer published§ in 1925:

* *Proceedings*, Am. Soc. C. E., March, 1922, Papers and Discussions, p. 529, Plate VII.

† "Continuous Mat Foundation for 22-Story Building," *Engineering News-Record*, July, 1922, pp. 73 *et seq.*

‡ By letter of December 4, 1925, to Maj. W. A. Danielson.

§ "Erdbaumechanik," by Charles Terzaghi, M. Am. Soc. C. E., Franz Deuticke, Wien, 1925.

(*a*) The relation between the diameter of the loaded area and the settlement produced by a given unit load depends essentially on the cohesion (actual shearing strength) of the soil. For soils with great cohesion the settlement produced by a given unit load increases in direct proportion with the diameter of the loaded area. On the other hand, for perfectly cohesionless soils, the size of the area has very little effect.

(*b*) With increasing depth of foundation, the settlement produced by a given unit load decreases. However, the ratio between the settlement, S_0, for a foundation depth of 0, and the corresponding settlement, S, for a depth, t, does not depend on the value of t alone, but on the ratio, $\frac{t}{d}$, between the depth of foundation and the diameter of the loaded area. Thus, if a foundation 5 ft. deep was found to reduce the settlement of a footing, 5 ft. square, by 50% of S_0, the effect of a footing 10 ft. square, and 5 ft. deep, in reducing settlement, will be very much less. In order that the potential settlement, S_0, of a footing 10 ft. square, and 0 ft. deep, may be reduced 50%, the depth would have to be increased to 10 ft., thus making the ratio, $\frac{t}{d}$, for both cases equal.

(*c*) The effect of the ratio, $\frac{t}{d}$, on the settlement is the less, the greater the cohesion. For perfectly cohesionless materials, a ratio of $\frac{t}{d} = 1$ (depth of foundation = diameter of loaded area) almost triples the bearing capacity and reduces the settlements to one-third of what they would be if the footing rested on the surface of the ground.

Knowing nothing more about soil mechanics than these three simple rules, the foundation engineer would at once discover that there are no contradictions in the data hereinbefore quoted. The experiments of the Austrian Committee and of Mr. Goldbeck were made on very cohesive soils. Hence, in strict agreement with theory, they showed that the settlement increases in direct proportion with the diameter of the loaded area. The Chicago tests were made on a cohesive soil, but the ratio between the depth of foundation and the diameter of the loaded area was twice as great for the larger area as for the smaller one. That fully accounts for the difference between the computed and the actual value of the settlement of the larger area. On the other hand, the soil of San Francisco belongs to the class of very slightly cohesive soils and, according to Rule (*a*), which is supported by experience, the size of the area should have comparatively little effect on the settlement.

Thus, if it combines actual observation with proper soil investigation and with a thorough consideration of the mechanical aspects of its problems, the same firm that gave up the attempt of systematic study in disgust, could gradually build up an experience covering the whole range of its activities and acquire a fairly reliable basis for the interpretation of the results of loading tests. However, under the present system of interpretation, loading tests are practically good for nothing.

Distribution of Soil Reactions Over Rigid, Loaded Slabs

The preceding discussion deals exclusively with the potential settlement of structures without questioning the stresses that may develop within the foundation. As a matter of fact, when referred to individual spread footings, these questions are of minor importance. However, the situation changes when it becomes necessary to connect the footings with each other, replacing the group of footings by a continuous mat. In order to keep the costs of such a mat within reasonable limits, the stresses must be computed as closely as possible. It is first necessary, however, to know the forces acting on the mat, and this is where difficulties arise. Suppose the mat is rectangular, and the dead load of the structure is distributed in such a manner that the resultant force passes through the center of gravity of the mat. Under such conditions, it is generally assumed, that the soil pressures are uniformly distributed over the entire area. However, according to the measurements carried out by M. L. Enger, M. Am. Soc. C. E., the distribution of the soil reactions over the base of a rigid slab is by no means uniform.* The pressures are equal to zero at the edge of the slab and greatest at the center, the pressure curve having a parabolic shape. In order to show the effect of the difference between uniform and parabolic stress distribution, the diagram, Fig. 3, has been plotted, showing the bending moments in a mat, loaded by four walls,

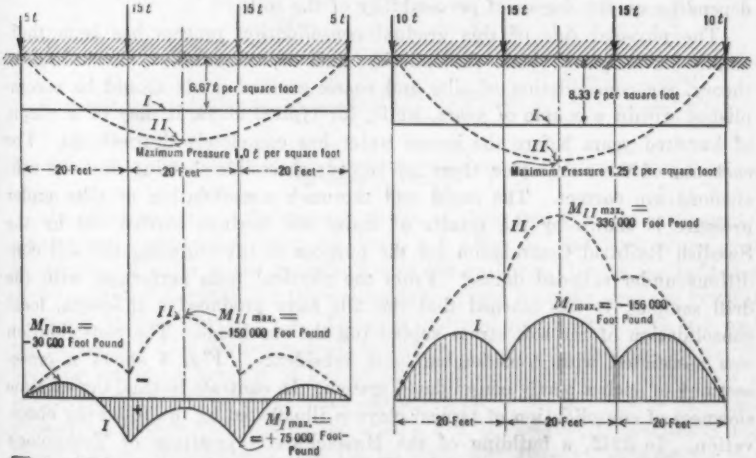

Fig. 3.—Bending Moments in Rectangular, Perfectly Rigid Slab for (I) Uniform and (II) Parabolic Distribution of the Soil Reaction.

and supported by a homogeneous soil. For a parabolic stress distribution the maximum bending moments will be about 100% greater than they are for a uniform reaction. Thus far, all attempts to deal theoretically with the problem of stress distribution have failed, and there is little hope for success within the near future. The data obtained by Professor Enger are confined

* "The Distribution of Pressure by Granular Material," *Engineering*, Vol. 101, 1916, p. 170.

strictly to perfectly cohesionless materials; hence, the information they furnish is not yet conclusive. Considering the importance of the possible error involved in assuming uniform stress distribution, the need of a more exhaustive experimental investigation of this phase of the foundation problem becomes obvious.

Effect of Degree of Permeability and of Time on Settlements

Another interesting aspect of the foundation problem concerns the changes that will take place in the soil under pressure. Since every soil, without exception, is compressible, the pressure tends to produce a decrease in volume. If the voids of the soil are filled with air, the volume change can take place at once, because the excess air can readily escape toward the surface. On the other hand, if the voids are completely filled with water, which is usually the case with very fine-grained and very compressible soils (silts, mud, clay), a decrease in volume obviously involves a considerable decrease in the water content, and the compression cannot possibly proceed with a greater speed than the corresponding speed of squeezing out the excess water. The less permeable the soil, the more slowly the water escapes, and, as a consequence, the more slowly the volume of the compressed soil decreases. Hence, the settlement of the foundation will not occur at once. There will be a lag, depending on the degree of permeability of the soil.

The physical side of this gradual consolidation process has been thoroughly investigated, both theoretically and experimentally. According to theory, the consolidation of silts and coarse-grained muds should be accomplished within a couple of years, while, for typical clays, it may be a couple of hundred years before the excess water has completely drained out. For each one of these examples, there are records accessible showing that the conclusions are correct. The rapid and thorough consolidation of silts under pressure is shown by the results of many test borings carried out by the Swedish Railroad Commission for the purpose of investigating the soil conditions under railroad dams.* From the physical tests performed with the drill samples, it was learned that the fills have produced a thorough, local consolidation of the soft strata supporting the surcharge. The consolidation was associated with considerable local subsidence. Fig. 4 shows a cross-section† of such a partly consolidated system. In contrast to this, the extreme slowness of consolidation of typical clays is illustrated by the following observation. In 1915, a building of the Massachusetts Institute of Technology was erected on a 30-ft. layer of sand, silt, and fill, resting on a soft clay deposit. The pressure exerted by the building on the clay amounts to approximately 1 500 lb. per sq. ft. In 1926, a test boring was made next to the heaviest section of the building. Physical examination of the drill samples disclosed the fact that the consolidation of the clay deposit had hardly started. A similar observation was published in 1923 by Thaddeus Merriman, M. Am. Soc. C. E., concerning the behavior of a mass of puddled clay forming part

* Statens Järnvägars Geotekniska Kommission, 1914–22, Stockholm, May, 1922.
† Copied from Statens Järnvägars Geotekniska Kommission, 1914-22.

of an embankment on the Ashokan Reservoir, Catskill Water Supply.* The surface of the puddled clay deposit, confined between the core-wall and the up-stream slope of the core ditch, carries the weight of an embankment 85 ft. high. Nevertheless, nine years after the construction of the embankment, a test boring disclosed the fact that the clay core was practically in its original condition.

The study of these and similar facts has led to the recognition of two essentially different types of settlements:

(*a*) Those due mostly to lateral flow, with very little, or no, consolidation; and,

(*b*) Those due to consolidation and lateral flow combined.

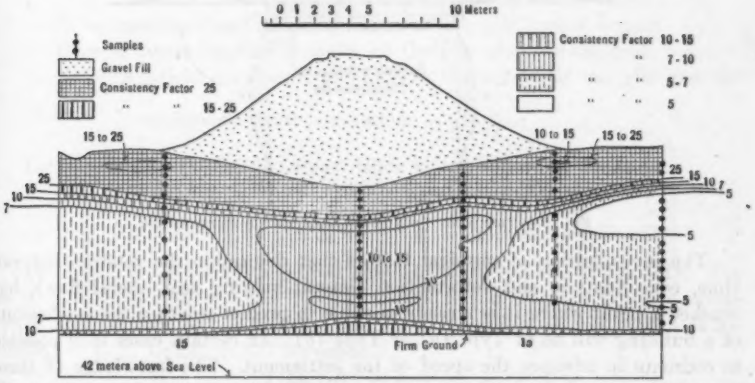

FIG. 4.—CONSISTENCY FACTORS OF SOIL DIRECTLY UNDER AND ON THE SIDE OF THE BANK AT SMEDSERÖD, SWEDEN.

In order to explain the characteristics of these two types of settlements, assume that two vertical reference lines, *a b* (Fig. 5), had been established in the ground, prior to the construction of the building. Under the influence of the weight of the building the soil tends to spread laterally, and the vertical lines, *a b*, become Curves *a c b*. The area included between the lines, *a b*, and the curves, *a b c*, is obviously equal to the area, $a\ a\ a_1\ a_1$, through which the building settles on account of lateral soil displacement. If, on account of a low permeability of the ground, the consolidation of the soil due to increased pressures, proceeds very slowly, the area, $a\ a\ a_1\ a_1$, represents practically the entire vertical displacement that the building will undergo, at least for one generation. The speed of the settlement will be governed exclusively by the laws of viscous flow in plastic materials, and if the flow is successfully stopped by driving deep sheet-piling, or by similar measures, the settlement of the building will be practically stopped. Those are the characteristics of Type (*a*) settlements.

On the other hand, if the material is permeable enough to release its excess water within a couple of years, the settlement of the building will consist of

* Discussion of the paper entitled "Design of Earth Dams," by Joel D. Justin, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXXVII (1924), p. 109.

two different parts, namely, the settlement through a space, $a\ a\ a_1\ a_1$, due to lateral bulging; and a settlement through an additional space, $a_1\ a_1\ a_2\ a_2$, due to compression of the soil beneath the loaded area. In this case the speed of the settlement will be governed by two essentially different laws: (1) Viscous flow in plastic materials; and (2) hydrodynamic stress compensation; which the writer formulated several years ago.* By artificially preventing the lateral flow of the material, one could somewhat reduce the settlements of the building, but one could not possibly prevent them. Those are the characteristics of Type (*b*) settlements.

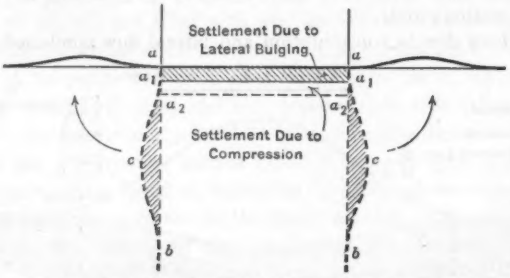

FIG. 5.—THE TWO PRINCIPAL SOURCES OF SETTLEMENT.

The investigation of physical factors that determine the relation between time, consolidation, and deformation (lateral bulging and lateral flow), has reached a point where it is already possible to predict whether the settlements of a building will be of Type (*a*) or Type (*b*). In certain cases it is possible to estimate in advance the speed of the settlement. The knowledge of these factors is particularly useful in all cases where the engineer faces the problem of reducing or stopping the settlement of an existing building.

BEARING CAPACITY OF INDIVIDUAL PILES

At the beginning of this paper it was assumed, that the engineer trusted with the design of a building, decided to erect it on spread footings. Let it now be assumed that his decision was premature. Suppose that test borings made after the first inspection of the building lot revealed the fact that the solid top layer of soil rested on a deposit of soft silt of such a thickness that there was no chance to carry the foundations down to solid ground. Then the engineer decided to put the foundations on piles. In order to find out how many piles he needed, he was obliged to drive a couple of test piles and determine the amount of load one individual pile could stand (unless he had previously tested some piles driven into similar deposits). To follow the traditional procedure, one must observe the average penetration of the pile under the last ten blows; compute the bearing capacity of the pile by the *Engineering News* formula; and divide the total weight of the building by the bearing capacity of the individual pile. This practice rests on two assumptions: (1) That the bearing capacity of a pile can be computed from

*"Die Theorie der Hydrodynamischen Spannungserscheinungen und ihr erdbautechnisches Anwendungsgebiet," *Proceedings*, International Congress for Applied Mechanics, Delft, Holland, 1924.

the effect of the blow; and (2) that the bearing capacity of the complete foundation is equal to the sum of the bearing capacities of the individual piles. To examine Assumption (1), let,

R = weight of the hammer.
G = weight of the pile.
L = length of the pile.
F = area of the cross-section of the pile.
E = modulus of elasticity of the pile material.
h = distance the hammer drops.
s = penetration produced by one blow.
m = coefficient of elasticity of the impact; $m = 0$ for perfectly non-elastic impact; and $m = 1$ for perfectly elastic impact.
Q_d = resistance against penetration of the pile, under impact.
Q = ultimate bearing capacity of the pile under static load.
C = empirical constant depending on the nature of the pile and the

$$\text{resistance against penetration} = \frac{Q_d L}{2 F E}$$

The theory of semi-elastic impact leads to the following equation:

$$Q_d = \frac{F}{L} E \left[-s \pm \sqrt{s^2 + \frac{2 R h}{E} \frac{R + m^2 G}{R + G} \frac{L}{F}} \right] \quad \ldots\ldots\ldots\ldots (3)$$

The value, m, is usually assumed equal to 0.5 (semi-elastic impact). For $m = 0$, the equation becomes Redtenbacher's formula, which is quite extensively used in Europe.* On the other hand, if perfect elastic impact is assumed, $m = 1$, Equation (3) becomes,

$$R h = Q_d s + \frac{1}{2} \frac{Q_d^2}{F} \frac{L}{E} = Q_d \left(s + \frac{1}{2} \frac{Q_d}{F} \frac{L}{E} \right) \quad \ldots\ldots\ldots\ldots (4)$$

or,

$$Q_d = \frac{R h}{s + \frac{1}{2} \frac{Q_d}{F} \frac{L}{E}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5)$$

If the fact that the term, $\frac{1}{2} \frac{Q_d}{F} \frac{L}{E}$, depending on both the nature of the pile and the resistance against penetration, is disregarded, and if this variable term is replaced by an empirical constant, C, independent of all these factors,

$$Q_d = \frac{R h}{s + C} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (6)$$

which is none other than the well-known *Engineering News* formula.

From a mechanical point of view, the assumptions on which these formulas are based, are sound, and there is not much doubt about their giving a fairly accurate conception of the resistance one has to overcome while driving the pile by a succession of impacts. Therefore, if experience shows that in certain cases the values furnished by the pile-driving formulas have practically

* "Formeln und Versuche über die Tragfähigkeit eingerammter Pfähle," by Ph. Krapf, Leipzig, 1906.

nothing in common with those determined by loading tests, being either far too small or far too large, the cause cannot be a defect in the formulas, but must be due to the fact that in these particular instances the forces, Q_d, resisting the penetration of the pile under impact, are fundamentally different from the forces, Q, which resist the penetration of the pile under static load.

This is precisely the situation a theoretical study of the pile-driving phenomenon has disclosed. It is known that the bearing capacity of piles depends on two different factors, namely, the frictional resistance acting along the sides of the piles, and the point resistance, or the resistance of the soil against being compressed and displaced by the pile. If these two resistances were dependent only on the character of the ground and on nothing else there could be no question about the resistance, Q_d, against driving the pile, and the bearing capacity, Q, of the pile, being identical. However, it is easy to prove that either one of them may be very different according to whether the pile is slowly forced down or driven by impact.

If a friction test is made on a layer of sand, by loading it and then measuring its resistance against shear, it is found that the shearing resistance of the loaded layer, immediately after the application of the load, is practically the same as it is three days later. If, however, precisely the same test is performed with a layer of clay immersed in water, it is found that immediately after application of the load the frictional resistance is very small, so small indeed that one has the impression that the material is lubricated. The full frictional resistance does not develop for a couple of days. If a laterally confined mass of sand is compressed, the speed with which the compression is performed has very little influence on the amount of work required to compress the material. On the other hand, the amount of work required for rapidly reducing the volume of 1 cu. ft. of laterally confined clay by 2 cu. in. may be 100 times as great as the amount of work required for producing the same volume change slowly. The physical causes of these phenomena are clearly understood. The writer has called them the hydrodynamic stress phenomena. They inevitably develop as a result of rapid application of loads or pressures on water-soaked materials with a low degree of permeability.* The theory has been repeatedly checked by experiment.

Applied to the mechanics of pile-driving, knowledge of the hydrodynamic stress phenomena has led to classifying soils in two main types. In certain materials (particularly in sand, gravel, and permeable artificial fills), the resistances acting while the pile is being driven, are practically identical with those acting on the pile under static load. Under such conditions the pile-driving formulas can be expected to furnish results of sufficient accuracy.

In other materials (very fine-grained silts, soft clays, etc.), the friction acting on the pile during the driving (hydrodynamic pile friction) is very much less than that which develops after a couple of days' rest (static pile friction), while the resistance of the point of the pile under impact (dynamic point resistance) is very much greater than its resistance under static load (static point resistance). Due to these facts the total resistance against penetration of the pile into such materials is:

* "Erdbaumechanik," by Charles Terzaghi, Wien, 1925.

(1) Dynamic resistance (resistance, Q_d, against penetration under impact), which is the sum of a very small frictional resistance (dynamic pile friction) and a very considerable point resistance (dynamic point resistance).

(2) Static resistance (ultimate bearing capacity, Q, under static load), which is the sum of a full frictional resistance (static pile friction) and a very small point resistance (static point resistance).

Since the pile-driving formulas furnish the value, Q_d, there is no assurance whatsoever that for this class of materials the value, Q, may be of the same order of magnitude. The value, Q_d, may, by chance, be equal to Q, the deficiency in static friction being compensated by an excess in point resistance; but this condition is by no means necessary. It could as well be very much greater or very much less, depending on the material. The following analogy demonstrates the error committed when applying the pile-driving formulas to resistance against the penetration of piles in this second class of materials. Suppose that a body slides on a rail under water. In order to produce a slow forward movement of this body, the only resistance there is to overcome is the static friction between the body and the rail, the resistance of the water being negligible. On the other hand, in order to keep the body sliding rapidly along the rail, the only resistance that counts is the resistance of the water, which, in this case, may be very considerable, while the frictional resistance along the rail will practically be eliminated because of a film of water trapped between the rail and the sliding body. Application of a pile-driving formula to materials of the second class is no more logical than an attempt to identify the static resistance of the sliding body against being set in motion with its dynamic resistance acting while moving rapidly through a viscous medium.

The best way to distinguish whether a material belongs to the first or to the second class is that of comparing the penetration per blow immediately before and after a period of rest of at least 24 hours. If these two penetrations are identical, one can be quite sure that the material belongs in the first class, and that the pile-driving formulas can be expected to furnish fairly reliable results. However, for this case, experience seems to show that the formulas based on the theory of impact (Equation (3)), furnish far better results than the *Engineering News* formula.

In 1925, the writer investigated the bearing capacity of an artificial fill, consisting of residual soil (stones, sand, and earth mixed), at Pasha Liman, on the Asiatic shore of the Bosphorus. When an attempt to make test borings with a normal outfit failed, several test piles were driven, one of which was loaded to the limit of its bearing capacity. The pile-driver was of the drop-hammer type with a weight of 0.575 metric tons. Fig. 6 (*a*) shows the results of the loading test; Fig. 6(*b*) shows the resistance against penetration under impact computed by the *Engineering News* formula (dotted lines) (Equation (6)) and by the theory of semi-elastic impact (thin full drawn line) (Equation (3)). The two sets of values are not very different, because the penetrations were rather important (ranging between 1.0 and 1.4 in.), although the values obtained by the impact theory are nearer the ultimate bearing capacity determined by the loading tests.

However, with decreasing values of the penetration produced by the last blows, the error involved in the *Engineering News* formula rapidly increases. A rather striking example can be quoted from pile-driving in Germany. As a result of many years of experience, the foundation engineers of Berlin, Germany, have developed the following empirical rule for the sandy soil of that city: For a pile that penetrates less than 0.4 in. under the impact of a 1-ton hammer, dropping from an elevation of 3.3 ft., the safe bearing capacity ranges between 20 and 25 tons. If the *Engineering News* formula is applied to this case, the ultimate bearing capacity is found to be 28.2 tons, corresponding to an allowable load of $\frac{1}{6} \times 28.2$ tons $= 4.7$ tons! For the ultimate bearing capacity of the same pile, Redtenbacher's formula* furnishes a value ranging between 52.8 to 63.6 tons, which is very close to the actual ultimate bearing capacity of these piles. On the other hand, by driving a 90-ft. wooden pile to refusal with a 5 000-lb. steam hammer (2.8-ft. drop), the safe bearing value of the pile, according to the *Engineering News* formula, should be equal to 127 tons. This is even more than the ultimate bearing capacity

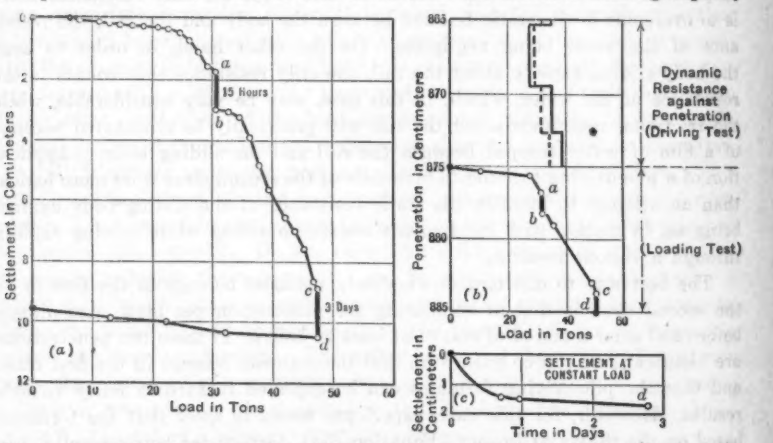

FIG. 6.—LOADING TEST PERFORMED ON TEST PILE IV ON APRIL 20–24, 1925, AT PASHA LIMAN, ASIA MINOR.

(92 tons) computed by Redtenbacher's formula. Thus, within the range of small penetrations, the defects of the *Engineering News* formula become strikingly apparent.

In connection with the pile tests performed in Pasha Liman, penetrations under the influence of a series of blows were observed, the hammer dropping from an elevation of 6.57 ft. (2 blows), 1.64 ft. (2 blows), 3.30 ft. (2 blows), 6.57 ft. (4 blows), and 9.83 ft. (4 blows). Fig. 7 shows the results of the observations. The points that correspond to a drop of the hammer of 6.57 ft. (200 cm.), 1.64 ft. (50 cm.), and 3.30 ft. (100 cm.), are located on a straight line. According to the theory of pile-driving by impact, this fact indicates that the resistance of the ground remained unchanged during the driving period.

* "Formeln und Versuche über die Tragfähigkeit eingerammter Pfähle," by Ph. Krapf. Leipzig, 1906.

Hence, for this period, a correct pile-driving formula ought to furnish identical values, irrespective of the height from which the hammer was dropped. Table 1 shows the values obtained by the *Engineering News* formula and the theory of semi-elastic impact, respectively. It demonstrates that for penetrations of less than 1 in., the *Engineering News* formula furnishes values that are by far too small, the error rapidly increasing with decreasing depth of penetration.

The obvious reason for this deficiency of the *Engineering News* formula is that the variable item, $\frac{1}{2}\frac{Q_d}{F}\frac{L}{E}$, (Equation (5)), forthcoming as a result of the theory of impact, has been replaced by a constant, C.

All these facts and data refer to typical materials of the first class, that is, to materials for which the pile-driving formula can be expected to furnish fairly reliable values. This conclusion was reached by observing, among other things, that the penetration produced by impact of a given intensity was practically the same before and after a period of rest.

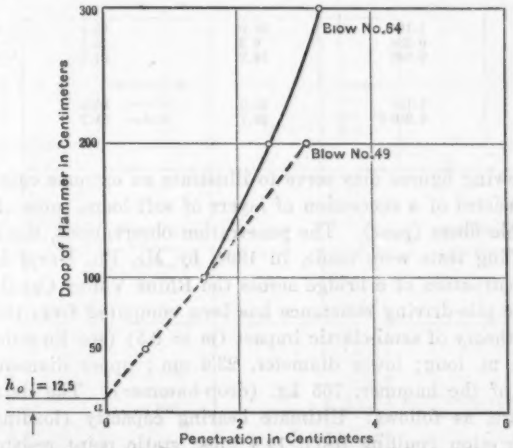

FIG. 7.—RELATION BETWEEN THE DROP, h, OF THE HAMMER AND THE CORRESPONDING PENETRATION, s, PRODUCED BY THE BLOW.

On the other hand, if the penetrations produced by a given impact before and after a period of rest are very different, the material belongs in the second class, and the values furnished by any pile-driving formula may be either equal to, very much larger than, or very much smaller than, the static bearing capacity of the pile.

Notice the physical processes associated with driving a pile into a material of the second class. Every blow of the hammer squeezes a certain quantity of water out of the soil beneath the point of the pile. The water escapes toward the surface through the space between the pile and the ground, and forms a film acting as a lubricant, just as any liquid does if trapped between two surfaces. Due to the presence of this film, the friction acting along the surface of the pile is very much less, whereas the force required to squeeze

the water rapidly out of the soil beneath the point of the pile is greater than that required to compress the same soil slowly. During a period of rest, the film of water is gradually absorbed by the soil and the full static pile friction develops. When pile-driving is resumed, one has to overcome both the static friction and the dynamic point resistance. However, during the same period of rest the annular space, serving as an outlet for the water squeezed out of the soil by impact, has closed. Hence, in certain kinds of soils, the dynamic point resistance acting after a period of rest may be much more than such resistance acting before the rest.

TABLE 1.—COMPARISON OF THE BEARING VALUE OF PILES.

Drop of hammer, in feet.	Penetration, in inches.	ULTIMATE BEARING CAPACITY, IN TONS, BY:		Remarks.
		Engineering News formula.	Semi-elastic impact theory.	
6.57	1.160	21.0	25.4	Should be equal
1.64	0.236	9.2	26.5	
3.30	0.590	14.2	24.6	
6.57	1.160	23.0	30.3	.....
9.83	1.280	29.7	35.2	.....

The following figures may serve to illustrate an extreme case of this type. The soil consisted of a succession of layers of soft loam, some of them mixed with vegetable fibers (peat). The penetration observations, the loading tests, and the pulling tests were made, in 1904, by Mr. Ph. Krapf in connection with the construction of a bridge across the Rhine Valley Canal in Austria.* The dynamic pile-driving resistance has been computed from the penetration data by the theory of semi-elastic impact ($m = 0.5$) (see Equation (3)). The pile was 8.4 m. long; lower diameter, 23.6 cm.; upper diameter, 33.3 cm.; and weight of the hammer, 765 kg. (drop-hammer). The data concerning this pile were, as follows: Ultimate bearing capacity (loading test), 17.2 tons; skin friction (pulling test), 14.4 tons; static point resistance, 17.2 — 14.4 tons = 2.8 tons; dynamic pile-driving resistance, computed from penetration after continuous driving, 20.0 tons; and dynamic pile-driving resistance, computed from penetration after a period of rest of 30 days, 80.0 tons.

In this case, by mere chance, the pile-driving formula applied to the effect of the hammer for continuous driving furnished a value close to the actual bearing capacity; while the value derived from the penetration after a period of rest was by far too great. In other cases, the second value is found to be closer to the actual bearing capacity. This may be learned from the customary practice of introducing into the *Engineering News* formula the values obtained after a period of rest.

Based on what is known about the physics of the penetration of a pile in the second class of material, the aforementioned data can be interpreted.

* "Formeln und Versuche über die Tragfähigkeit eingerammter Pfähle," by Ph. Krapf, Leipzig, 1906.

While continuously driving the pile into the ground, the skin friction is practically eliminated. Hence, the dynamic pile-driving resistance is practically equal to the dynamic point resistance; or, before the period of rest, the dynamic point resistance is 20.0 tons, as compared with 2.8 tons static point resistance.

After a 30-day period of rest, the hammer had to overcome the full static friction (17.2 tons) plus the dynamic point resistance (62.8 tons); or a total of 80 tons.

Fig. 8 represents this interpretation graphically. The shaded areas correspond to the pile friction acting during the pile-driving process (*a*); under impact after a period of rest (*b*); and under static load (*c*).

This and similar examples show that the application of any pile-driving formula to the bearing capacity of piles driven into the second class of materials is a gamble, trusting that the deficiency in skin friction associated with the driving of the pile may, by chance, be compensated by the corresponding excess in point resistance.

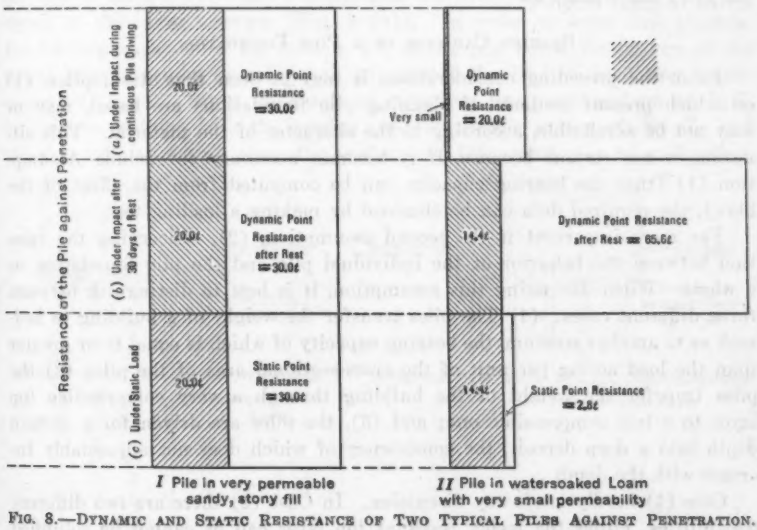

FIG. 8.—DYNAMIC AND STATIC RESISTANCE OF TWO TYPICAL PILES AGAINST PENETRATION.

Considering these facts it seems to be futile to attempt any further improvements in the field of pile-driving formulas. For materials of the first class, the pile-driving formulas were good enough even fifty years ago; while for materials of the second class, no reliable pile-driving formulas are possible at all, because the resistance against penetration of the piles under impact into such materials has physically nothing more in common with the static bearing capacity than has the static friction between solids (following the law of Coulomb) with the viscous resistance of liquids against rapid deformation (following the law of Newton). In the future, the penetration curve of piles, and particularly the difference between the rate of penetration before and

after a period of rest, may give more reliable information about the character of a ground of the second class than a test boring, but it never will give any reliable information about the bearing capacity of the piles.

Therefore, future investigations in the field of bearing capacity of piles should be directed toward studying the effect of the shape, thickness, and length on the bearing capacity of piles driven into different kinds of soils. In this connection attention should be called to the exhaustive investigations by the engineers of the Whangpoo Conservancy Board, in Shanghai, China. The tests included more than forty complete driving, loading, and pulling tests in soils the nature of which was previously investigated by test borings. The soil consisted principally of clay and silt, apparently belonging to the second class of materials. The tests furnished valuable information concerning the effect of the shape and length of the piles on the ultimate bearing capacity. Characteristically enough, the published reports do not contain any information on the penetration observations, for the reason that "the application of general pile-driving formulas to the local conditions has resulted in great errors".*

Bearing Capacity of a Pile Foundation

From the preceding considerations it may be seen that Assumption (1) on which present methods of planning pile foundations are based, may or may not be admissible, according to the character of the material. This situation is not serious because, if it becomes necessary to exclude Assumption (1) (that the bearing capacity can be computed from the effect of the blow), the required data can be obtained by making a loading test.

Far more important is the second assumption, (2), concerning the relation between the behavior of the individual pile and the pile foundation as a whole. When discussing this assumption, it is best to distinguish between three different cases: (1) The piles transfer the weight of a building to bedrock or to another stratum, the bearing capacity of which is equal to or greater than the load acting per unit of the cross-sectional area of the pile; (2) the piles transfer the weight of the building through a very compressible top layer to a less compressible one; and (3), the piles are driven for a certain depth into a deep deposit, the consistency of which does not appreciably increase with the depth.

Case (1) hardly needs any discussion. In Case (2) there are two different possibilities; either the feebly resistant top layer can or cannot be solidified by pile-driving. For instance, the average water content of the soft clay deposits in Boston, Mass., ranges between 30 and 40%, corresponding to a volume of voids ranging between 45 and 52%, filled with water. If piles are driven into a deposit of this kind, the surface of the clay rises between them through a height of several inches, which indicates that the volume of voids of the material remains practically unchanged. Pile-driving produces not only no consolidation of the deposit, but it even seems to cause the material to become softer. This last conclusion has been drawn from the observation

* Whangpoo Conservancy Board, S. H. T. Series 1, No. 7. Various reports to the Engineer-in-Chief on special investigation, Shanghai, 1921; report to the Engineer-in-Chief on pile tests.

that the compressive strength of an undisturbed sample of clay is always considerably greater than that of the same sample, with the same water content, after the sample has been moulded by external pressure. In striking contrast to this behavior of the deposits of blue clay stands the behavior of other very fine-grained deposits. Quite recently, when piles were driven into a deposit of exceedingly fine-grained saturated quicksand, the surface of the deposit subsided between the piles through a distance of almost 1 ft. Judging from previous experience, the original volume of voids of the material was certainly not greater than 45%, corresponding to the volume of voids of a clay with a water content of 30 per cent. Nevertheless, based upon the subsidence of the surface of the deposit, the pile-driving must have reduced the volume of voids by several per cent., which, in turn, involves a considerable increase in the bearing power and a decrease of the compressibility of the deposit.

In these two cases, the piles must serve two very different purposes if they are to be utilized to full advantage. In the first case, they should transfer the load to the more resistant sub-stratum, thus diverting the pressure from the upper to the lower stratum (Fig. 9 (*a*)). In order to serve this purpose, the bearing capacity of the section, *b c*, of the pile below the bottom of the upper layer must be great enough to support the load without any appreciable settlement. Since the result of a loading test performed on an individual pile includes both the bearing capacity of this section of the pile and the skin friction acting along the section, *a b*, this result may be misleading.

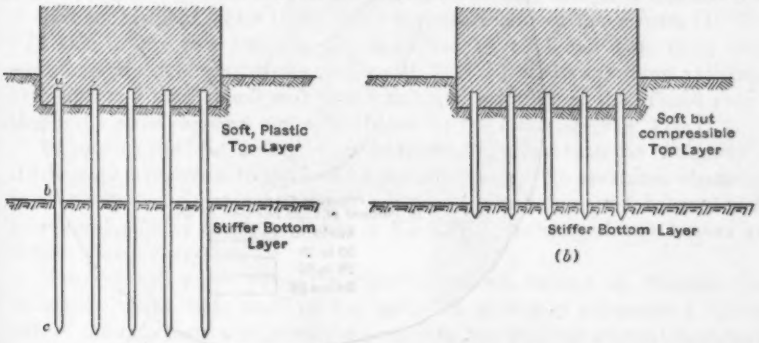

FIG. 9.—PILE FOUNDATION ON A TWO-LAYER SYSTEM.

Fig. 9 (*b*) represents the same combination of strata as Fig. 9 (*a*), with the only difference that the top layer is apt to be solidified (artificial fill, quicksand, etc.). In this case it may be more economical to use the piles merely for the purpose of increasing the bearing capacity of the top layer by pile-driving, letting the piles extend to the bottom of the top layer only. Here, again, the result of a loading test performed on an individual pile, prior to driving the others, would fail, by far, to furnish any reliable information concerning the number of piles required. What is needed is to learn about the effect of driving on the density of the surrounding soil. Deposits that can be consolidated by pile-driving belong almost exclusively to those

classes of material for which the pile-driving formulas are valid. The proper procedure would be to start with piles far apart and to drive intermediate piles until the bearing capacity, computed from the penetration, indicates the resistance of a well-consolidated ground.

In the third case (piles driven into the upper part of a very deep deposit with a fairly uniform consistency), the value of the piles may or may not be problematical, depending on the ratio between the width of the foundation and the length of the piles. Suppose that the pressure diagrams, Fig. 10 (*a*) and Fig. 10 (*b*), have been plotted. The left-hand sections of these diagrams show the distribution of soil pressures beneath two raft foundations with different widths, computed approximately by Boussinesq's theory. The right-hand sections of the same diagrams show the change in the stress distribution due to the presence of 20-ft. piles in the ground, computed by the same theory.

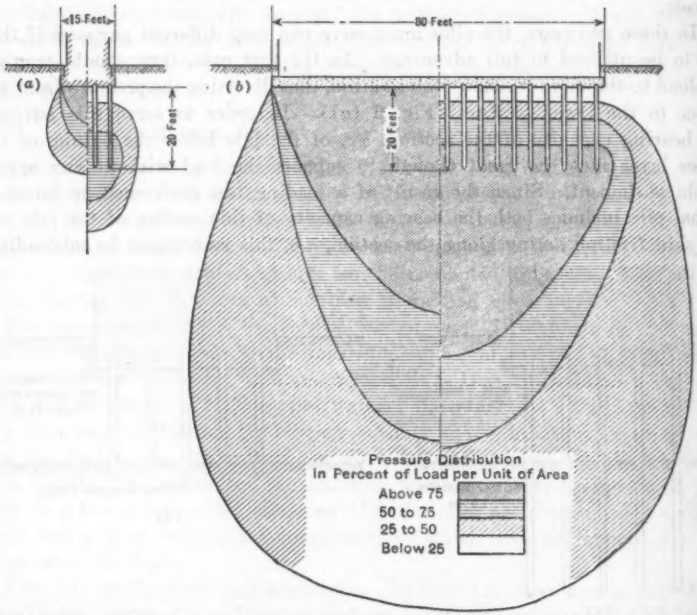

FIG. 10.—STRESS DISTRIBUTION BENEATH A NARROW (*a*) AND A WIDE (*b*) PILE FOUNDATION.

If the length of the piles is at least equal to the width of the base, their effect is obviously very beneficial (Fig. 10 (*a*)). The piles reduce the intensity of the maximum pressure acting on the ground, and, in addition, they shift the zone of maximum stress from the surface to the level where their lower ends are located. Since the effect of the depth of foundation on the bearing capacity of the ground depends, as previously noted, not on the depth, *t*, but on the ratio between the depth of foundation and its width, the effect of transferring the pressure to a deeper level, in the case of Fig. 10 (*a*), may be quite important. On the other hand, if the width of the foundation is considerably greater than the length of the piles (Fig. 10 (*b*)),

the pressure-reducing effect of the piles is very small and the ratio between the depth of foundation and the width of the structure is also very much smaller than in the case of Fig. 10 (*a*). Hence, the beneficial effect of the piles may be negligible and the money invested in them may represent an unwarranted expenditure. The reason piles are so generously used beneath raft foundations, in spite of the facts represented in Fig. 10, is that many engineers confound the bearing capacity of the individual piles with the bearing capacity of a score of them. Yet one can hardly conceive of a more obvious fallacy. The individual pile spreads the load over a wide area, thus reducing the specific soil pressure to a negligible item, while beneath a wide pile foundation, the soil pressure is just as great as it is under a simple raft foundation of equal width. In structural engineering such a procedure would correspond to considering that, since a bridge has stood the load of a single car without measurable deflection, it will stand the load of 200 cars, acting simultaneously.

Some years ago a company intended to construct a power house with a raft foundation supported by 500 reinforced concrete piles, 25 ft. long, because a similar building resting without piles on the same mud deposit had suffered a subsidence of more than 12 in. A loading test performed on an individual pile showed that a weight of 4 tons did not produce a measurable sinking of the pile. As a consequence, it was believed that, if a load of not more than 2 tons were assigned to each pile, no settlements would occur. After the mud deposit was investigated, the company was advised to sell the piles and construct a simple raft foundation, because conditions were similar to those shown in Fig. 10(*b*). However, as some of the piles were already driven, it was decided to drive all of them. Fig. 11(*a*) shows the settlements of the corners (I, IV, VII, X), of the new building (*a*) supported by piles and Fig. 11(*b*) the simultaneous settlements of the corners (*A, B, E, F*) of the adjoining building (*b*), supported by a simple raft foundation and exerting on the ground practically the same pressure per unit of area as the building (Fig. 11(*a*)).

By keeping the load, acting on the individual pile, less than the "safe load", it is merely transferred to the level of the pile points. No assurance whatever is obtained as to how the ground below the pile points will behave, and in cases of piles of the type shown in Fig. 10(*b*), the ground may behave as if they were non-existent.

Hence, in all cases where the piles do not act strictly as columns, the knowledge of the "safe load" of the individual piles only represents a minor part of the information required for predicting the behavior of the foundation as a whole. The essential part of the problem consists of studying the effect of the load on the ground located around the piles and beneath the points. Yet building codes are satisfied with specifying the load per pile, disregarding the true cause of future complications.

Effect of Freezing on Foundations

During recent years valuable contributions have been made to current knowledge of the effect of frost in the soil and on the structures supported by it. Most important among them are the investigations of Professor Stephen Taber,* of the University of South Carolina, concerning the capacity of

* "The Growth of Crystals Under External Pressure," *American Journal of Science*, 4th Series, Vol. 41, pp. 532–556; "Pressure Phenomena Accompanying the Growth of Crystals," *Proceedings*, National Academy of Sciences, Vol. 3, No. 4, pp. 297–302.

freezing veins of water to absorb additional water out of the surrounding clay soil, or through the clay soil from the ground-water, and to develop from thin seams into layers with a considerable thickness. The space required for the growth is produced by the pressure of crystallization, forcing the soil out of the way.

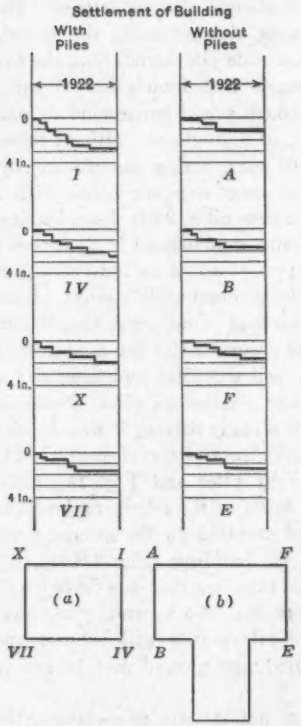

FIG. 11.—SETTLEMENT OF TWO ADJOINING BUILDINGS.

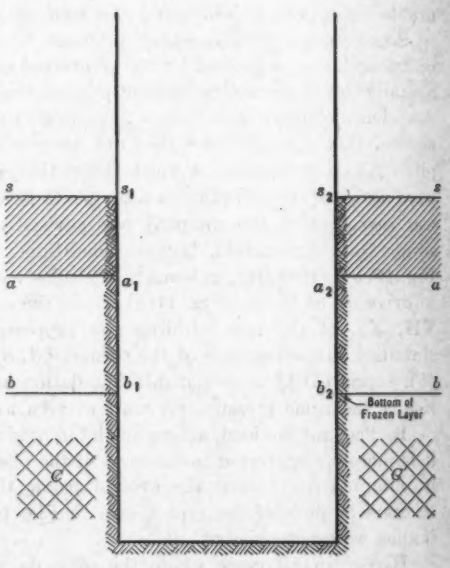

FIG. 12.—EFFECT OF SOIL FREEZING ON A FOUNDATION PIER.

In connection with foundation engineering, the importance of frost action essentially resides in the capacity of the freezing ground to lift foundations bodily several inches above their original position. This is apt to cause differential heaving associated with secondary stresses in the superstructure. Quite a number of such cases have been brought to the attention of the profession and many more have been undoubtedly observed.*

* "Freezing Ground Acts Like Hydraulic Jack: Heaving and Settling Back of Piers of Railroad Bridge Over City Street, Lead to Interesting Deduction," by H. J. Gilkey, *Engineering News-Record*, Vol. 79 (1917), pp. 360–361; "Some Observations on Effect of Frost in Raising Weight," by L. B. Wyckoff. *Engineering News-Record*, Vol. 80 (March 28, 1918), pp. 627–628; "Water Expansion in Ground Cause of Heaving in Winter," by C. D. Norton, *Engineering News-Record*, Vol. 80 (May 30, 1918), p. 1058; "Action of Frost in Heaving Concrete Piers," by L. DeB. McCready, *Engineering News-Record*, Vol. 91 (August 30, 1923), p. 360.

However, thus far, apparently no attempt has been made to go beyond the observed phenomena and to analyze the conditions required for producing the lifting effect. The following conclusions represent the results of a preliminary survey of these conditions.

The freezing starts at the surface of the ground and gradually proceeds toward the interior. At the same time the soil freezes to the pier, the adhesion being practically equal to the shearing strength of the frozen soil. Suppose that at some intermediate state, freezing has proceeded from the surface, *S-S* (Fig. 12), to the level, *aa*, the frozen soil being cemented to the pier along the faces, a_1 s_1 and a_2 s_2. If the freezing process continues still farther, causing the freezing and the expansion of the soil located within the space, *a a b b*, there are then three possibilities:

(*a*) The ground beneath the freezing layer is feebly compressible, and the adhesion between the faces, $a_1 s_1$, $a_2 s_2$, and the soil may be greater than the weight of the pier. In this case the pier should be lifted bodily, as freezing proceeds, leaving an empty space beneath it.

(*b*) The ground is compressible, but the adhesion along the faces, a_1 s_1 and a_2 s_2, is greater than the weight of the pier. In this case, with the pier as a reaction, the freezing soil will exert a downward pressure causing a small, but permanent, consolidation of the ground within the spaces, *C-C*.

Every frost season will add some additional consolidation, until, after several seasons, the soil located within *C-C* will be so compact, that finally a frost will succeed in lifting the pier, the consolidated material, *C-C*, acting as a reaction.

(*c*) The ground is feebly compressible, but the weight of the pier is greater than the adhesion. In this case the expansion of the soil, *a a b b*, will cause a slow upward creep of the soil, *s s a a*, along the outside of the pier, associated with an upward lift equal to the adhesion. The result should be a fatigue effect, similar to repeated application and removal of a live load, causing slight additional settlements of the pier.

From these remarks it may be learned that the lifting action of the expanding ground should depend on several factors, namely, the compressibility of the unfrozen foundation soil, the compressive strength of the frozen soil, the depth of freezing, and, finally, on the ratio between the weight of the pier and the total adhesion between the pier and the frozen ground. As experiences increase, other factors may have to be added to this list. Considering the scarcity of available information, any observations made in this field may increase the engineer's capacity for predicting the effect of freezing on proposed foundations.

Soil Classification Based on Elastic Constants of Soils

The preceding parts of this paper have brought out the following facts: (1) The settlement produced by a given unit load may increase either in direct proportion with the diameter of the loaded area, or at a very much smaller rate, depending on the character of the soil; (2) the settlement of a building may be due to volume change combined with lateral flow, or to lateral flow alone, depending on the character of the soil; (3) the pile-driving

formulas may furnish fairly reliable or utterly inconsistent results, depending on the character of the soil; and (4) the driving of piles into a soft top layer may cause a softening of the soil associated with a rise of the surface, or consolidation associated with subsidence, depending on the character of the soil.

The fundamental requirement for bringing the manifold foundation experience into a rational working system consists of establishing a system for the classification of soils based essentially on those characteristics that are of engineering importance.

Thus far, attempts to classify soils (including the revised soil classification* scheme of the Special Committee on the Bearing Value of Soils for Foundations, etc., of the Society) have been based essentially on such properties of soils as: (*a*) mineral composition; (*b*) volume of voids; (*c*) grain composition (result of mechanical analysis); (*d*) water content; and (*e*) percentage of colloidal material present in the soil. A study of these factors, covering a period of several years, has disclosed that:

(*a*) The mineral composition of very fine-grained soils cannot be determined except by elaborate optical or chemical methods, the cost of which is forbidding. Even if it were possible to make such a determination, the benefit would be doubtful.

(*b*) The volume of voids is so complicated a function of the shape and uniformity of the grains that it is impossible to correlate it with any definite properties of the soil, even if both the uniformity and the effective size of the material are known.

In 1926, the writer investigated the permeability of two sets of samples; one coming from a deposit of modified glacial drift near Westfield, Mass., and the other from a glacial lake deposit near Springfield, Mass. The effective size of both materials ranged between 0.01 mm. and more than 1 mm. In order to simplify the laboratory work, an effort was made to determine for each set the relation that exists between the uniformity coefficient (according to the well known definition of Allen Hazen, M. Am. Soc. C. E.) and the volume of voids. Fig. 13 shows the results of this investigation. Although there was a slight difference only between the shape of the grains and the mica content of the various samples, the relation between the volume of voids and the uniformity coefficient was found to be very erratic. To quote an example: At a voids' ratio of 0.8 (volume of voids of 45%), Soil No. 7, from Westfield, is as compact as it could possibly be and, if loaded, it would have a considerable bearing capacity. At the same voids ratio of 0.8 (volume of voids of 45%), Soil No. 34, from the same locality, would be very compressible, although its uniformity coefficient is practically equal to that of Soil No. 7.

(*c*) A mechanical analysis, according to the revised size grades of the Society's Special Committee on the Bearing Value of Soils for Foundations, etc., involves more than two weeks of work per sample, and the costs are forbidding. Nevertheless, the writer performed more than one hundred complete

* *Proceedings*, Am. Soc. C. E., February, 1921, Papers and Discussions, p. 17.

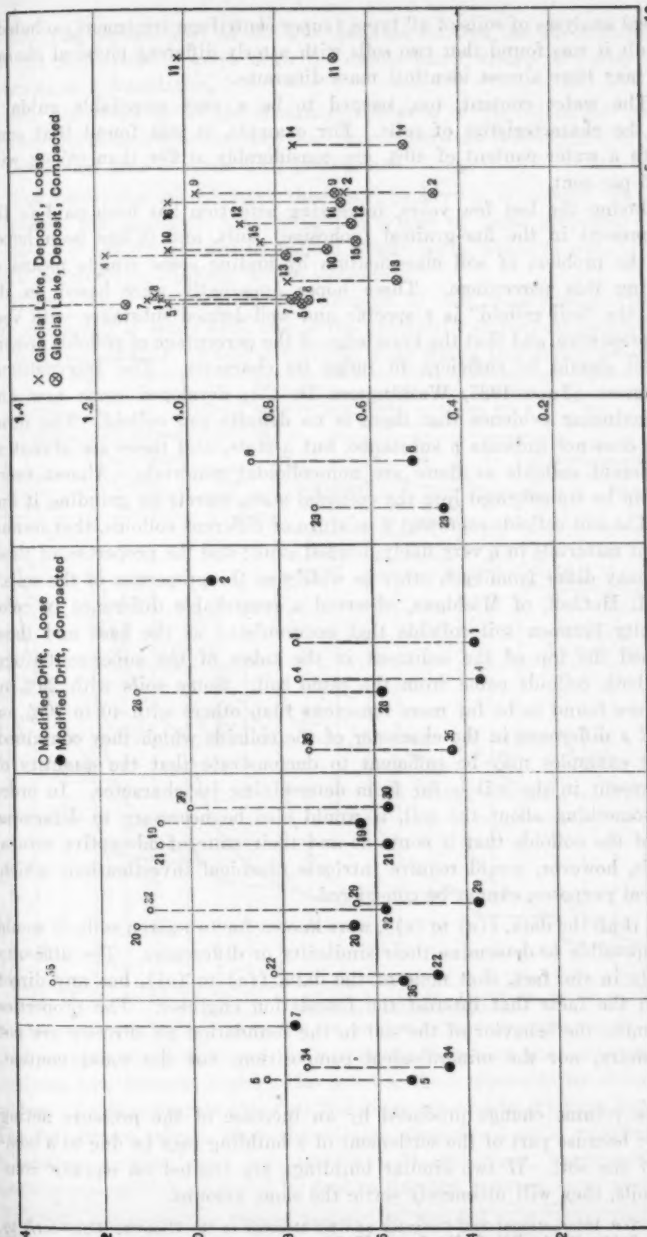

FIG. 13.—RELATION BETWEEN VOIDS RATIO AND UNIFORMITY.

mechanical analyses of soils of all types (super-centrifuge treatment excluded). As a result it was found that two soils with utterly different physical characteristics may have almost identical mass diagrams.

(*d*) The water content, too, seemed to be a very unreliable guide in judging the characteristics of soils. For example, it was found that some clays with a water content of 40% are considerably stiffer than others with one of 25 per cent.

(*e*) During the last few years, increasing attention has been paid to the colloids present in the fine-grained (cohesive) soils, and it has been hoped to solve the problem of soil classification by finding some simple means of determining this percentage. These hopes apparently were based on the idea that the "soil colloid" is a specific and well-defined substance with very unusual properties, and that the knowledge of the percentage of colloids present in the soil should be sufficient to judge its character. The International Soil Congress (June 1927, Washington, D. C.), developed some new and rather convincing evidence that there is no definite soil colloid. The term, "colloid", does not indicate a substance, but a state, and there are almost as many different colloids as there are non-colloidal materials. Almost every mineral can be transformed into the colloidal state, merely by grinding it fine enough. The soil colloids represent a mixture of different colloids, that means, of different materials in a very finely divided state; and the properties of these mixtures may differ from each other as widely as the properties of the soils.* Mr. M. M. McCool, of Michigan, observed a remarkable difference in color and tenacity between soil colloids that accumulated at the base and those that formed the top of the sediment in the tubes of the super-centrifuge, although both colloids came from the same soil. Some soils with 20% of colloids were found to be far more tenacious than others with 40 to 70%, on account of a difference in the character of the colloids which they contained. These few examples may be sufficient to demonstrate that the quantity of colloids present in the soil is far from determining its character. In order to learn something about the soil, it would also be necessary to determine the type of the colloids that it contains and their state of adsorptive saturation. This, however, would require intricate chemical investigations which, for practical purposes, cannot be considered.

Hence, if all the data, ((*a*) to (*e*)), were known for two given soils, it would still be impossible to determine their similarity or differences. The difficulty is obviously in the fact, that none of the data, ((*a*) to (*e*)), has any direct bearing on the facts that interest the foundation engineer. The properties that determine the behavior of the soil in the foundation pit directly are not the uniformity, nor the mineralogical composition, nor the water content. They are:

(1) The volume change produced by an increase of the pressure acting on the soil; because part of the settlement of a building may be due to a compression of the soil. If two similar buildings are erected on equally compressible soils, they will ultimately settle the same amount.

* "The First International Soil Congress and Its Message to the Highway Engineer," by C. Terzaghi, *Public Roads*, Vol. 8, No. 5, pp. 89–94.

(2) The permeability of the soil; because the less the permeability of the soil the more time it takes until the excess water drains out after the construction of a foundation.

(3) The cohesion or the shearing resistance of the soil under zero load; because the cohesion determines the relation between settlement and diameter of the loaded area at equal unit pressures.*

The determination of these three soil properties is enough for practical purposes. The question concerning the causes of these properties—whether or not they are determined by colloid constituents, effective size, or mineralogical composition—belong in the laboratory.

Hence, when approaching the problem of soil classification, the writer first attempted to study each one of the practically important properties, ((1) to (3)), individually, to find out within what limits they could possibly vary.

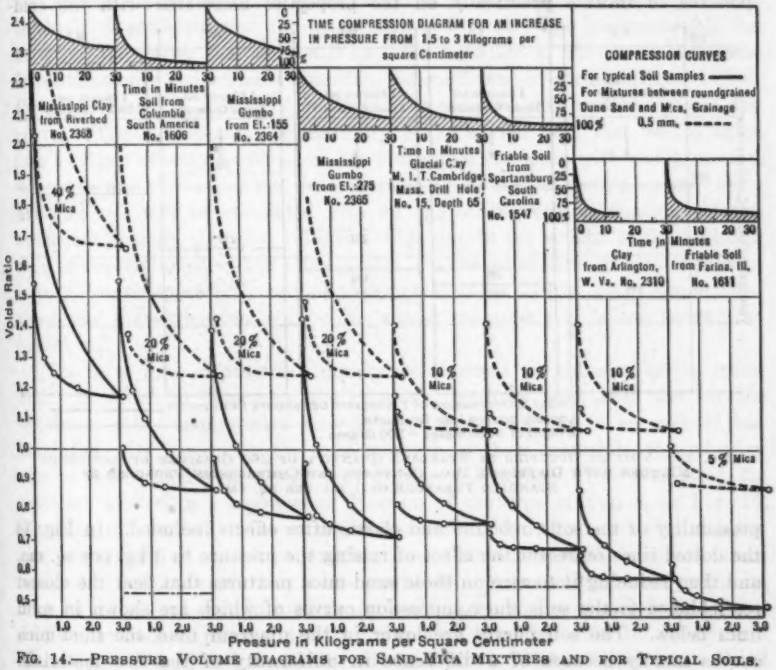

FIG. 14.—PRESSURE VOLUME DIAGRAMS FOR SAND-MICA MIXTURES AND FOR TYPICAL SOILS.

A careful study disclosed the surprising fact that the compressibility of soils may vary between limits as far apart as the compressibility of concrete and rubber. To demonstrate this fact the diagrams of Fig. 14 have been plotted. The full drawn curves show the volume changes of soils, ranging between Mississippi gumbo (most compressible soil) and clean, round-grained sand (least compressible), produced by first raising the pressure from zero to 3 tons

* "Erdbaumechanik," by Charles Terzaghi, M. Am. Soc. C. E., Franz Deuticke, Wien, 1925.

per sq. ft. (3 kg. per sq. cm.), and then gradually reducing the pressure to zero. At the outset the water content of each one of the samples was equal to the water content of the material after slow sedimentation in quiet water (Atterberg's lower liquid limit). From Fig. 14 it may be learned that the compressibility of the sand amounts to only a small fraction of the compressibility of the Mississippi gumbo. Even a year ago (1925) the cause of the tremendous difference in the compressibility of different soils was not yet quite clear to the writer. Suspecting that it might be due to the greater or smaller abundance of scale-like particles in the soil, he induced Glennon Gilboy, Jun. Am. Soc. C. E., of the Massachusetts Institute of Technology, to investigate the elastic properties of differently proportioned mixtures of sand and mica, both with a grain size of 0.5 mm.* The results of these investigations certainly were striking. By properly selecting the mica content of the sand, it was possible to imitate practically all the properties associated with the com-

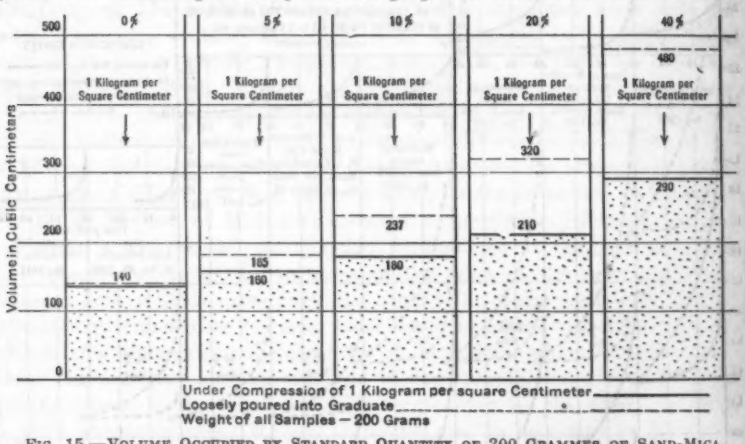

FIG. 15.—VOLUME OCCUPIED BY STANDARD QUANTITY OF 200 GRAMMES OF SAND-MICA MIXTURE WITH DIFFERENT MICA CONTENTS AND COMPRESSION PRODUCED BY STANDARD PRESSURE OF 1 KG. PER SQ. CM.

pressibility of the soils, rebound and elastic after effects included. In Fig. 14 the dotted lines represent the effect of raising the pressure to 3 kg. per sq. cm. and then reducing it to zero on those sand-mica mixtures that bear the closest resemblance to the soils the compression curves of which are shown in solid lines below. The soil curves are lower in the diagram than the sand-mica curves merely because of a difference in uniformity of the two materials. In order to make these facts clearer, the diagrams, Fig. 15, have been prepared. The upper row of numbers shows the space occupied by a standard quantity of 200 grammes of sand-mica mixtures with a mica content of 0%, 5%, 10%, 20%, and 40%, respectively. The lower row shows the space occupied after compression produced by a standard pressure of 1 ton per sq. ft.

Thus, it becomes evident that one of the most important properties of the soils—compressibility—has nothing to do with the effective size, the uni-

* *Proceedings*, Am. Soc. C. E., February, 1928, Papers and Discussions, p. 555.

formity, or the colloid content. It is merely the mechanical effect of a greater or smaller abundance of scale-like particles.

The property next in importance concerns the permeability of soils. The first tests that the writer made for determining the permeability of very fine-grained materials, such as clays, lasted several months.* However, a method has been devised, based on the theory of hydrodynamic stresses, by which the coefficient of permeability can be determined from not more than one dozen readings. By using the same sample that serves for investigating the compressibility, the determination can be made in 24 hours.† The test is made by first raising the pressure which acts on the sample by 100 per cent. The less permeable the material, the longer will be the time during which the excess water is squeezed out. The gradual loss of water betrays itself by a gradual settlement. By observing the settlement in specified time intervals covering a period of 24 hours, a curve of settlement against time may be obtained, as shown near the upper edge of Fig. 14. The less permeable the material, the flatter the curve will be. The equation of the curve is known. It contains only one variable quantity, the coefficient of permeability. Therefore, it is merely a matter of arithmetic to compute this coefficient from the curves. The coefficient of permeability is the second item that has to enter into a system of soil classification. It determines the speed with which the soil will settle and whether or not the settlement of a building resting on very compressible soil will be associated with an appreciable volume change. If the coefficient is high, the total volume change due to the weight of the building may be accomplished within a few years. On the other hand, if this coefficient is very low, no appreciable volume change may occur within one or two generations, and the settlements will be due almost exclusively to lateral flow of the loaded soil.

The third item of practical importance concerns the cohesion of the material. In order to determine the cohesion, methods have been devised for obtaining drill samples with their original water content in an undisturbed condition. By measuring the cube strength of these materials, the cohesion can be determined with a sufficient degree of accuracy. The results of the tests are plotted in a consistency diagram of the type of Fig. 4, or Fig. 16. By using this method of investigation it was found that there are clay deposits that are still in an undrained condition, which means that their consolidation under the influence of their own weight is still proceeding. As examples may be mentioned certain mud deposits along the shores of the Golden Horn in Constantinople; parts of the blue clay deposit which underlies Boston and Cambridge, Mass.; and a clay deposit in Detroit, Mich. There is no doubt that many others of a similar kind exist. The surface of such deposits would gradually subside even if no buildings were erected on them, and the construction of buildings accelerates the process, involving considerable settlements regardless of the type of foundation.

* "Die physikalischen Grundlagen des Technisch-geologischen Gutachtens," by Charles Terzaghi, M. Am. Soc. C. E., *Zeitschrift* des Oesterreichischen Ingenieur- and Architekten Vereins, September, 1921.

† "Die Berechnung der Durchlässigkeitsziffer der Tone aus dem Verlauf der hydrodynamischen Spannungserscheinungen," *Sitzber.* der Akad. der Wiss. in Wien, Math. Natur., Abt. IIa, 1923.

The first two items, compressibility and permeability, determine the type of soil with a greater precision than the concrete term, "1 : 2 : 4", describes the property of artificial stones. A third item, cohesion (shearing resistance) for clays, and density for sands and quicksands, determines the state in which the soil occurs. The methods for measuring the density (firmness) of sandy materials have not yet been developed, but there seems to be no insurmountable obstacle to the solution of the problems involved.

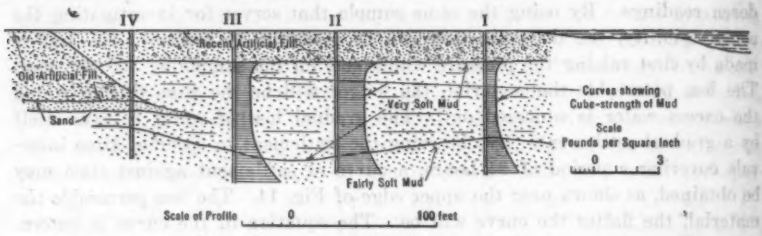

FIG. 16.—CONSISTENCY DIAGRAM FOR A NEW MUD DEPOSIT AT AÏWAN SERAÏ, NEAR CONSTANTINOPLE, TURKEY.

With these three items as a basis, a soil classification could be devised which would serve to bring soils into as rational a system as that which covers artificial construction materials, because soils with equal compressibility, equal permeability, and occurring in the same state, will behave, under load, in an identical manner, regardless of what the ultimate causes of their properties may be.

However, in practice, there are two serious difficulties associated with any attempt to classify soils according to the method proposed in the preceding discussion. The first is the difficulty of obtaining undisturbed samples of certain soils. According to the results obtained by Swedish investigators, disturbing a soil without changing its volume of voids and its water content may cause the cohesion (consistency) of the soil to be reduced by an amount ranging between 75 and 96% of its original value. Hence, the conclusions derived from testing the consistency of a drill sample may be very misleading unless one has succeeded in securing the sample in an undisturbed state.

The second difficulty is due to the fact that the character of the natural ground varies to a greater or less extent from foot to foot, in a horizontal and in a vertical direction, the variations depending on how the soil deposit was formed. In order to get an accurate conception of the average character of the soil, a considerable number of samples ought to be tested. The tests required for determining the constants mentioned in the preceding discussion are too expensive, in time and money, to be performed on every individual sample. Hence, efforts are made to work out certain simple routine tests, the results of which would furnish the approximate information desired at a reasonable amount of time and labor.

CONCLUSIONS

Considering the present state of knowledge in the field of soil mechanics, the prospects concerning the future of foundation engineering as an applied

science are decidedly encouraging, the principal obstacles against progress in this field having been removed. The elastic properties of the most troublesome soils, clays included, are now at least as thoroughly known as those of concrete or steel. The methods of soil classification and soil identification have reached a point where it can be clearly seen what is needed for identifying soil materials. The relations between the size of the loaded area, depth of foundation, character of the soil, and intensity of the load are well analyzed, at least in principle, and the physics of the time effects are clearly understood. Yet, stress must be laid on the fact that the knowledge thus obtained merely serves as a means for uprooting certain persistent prejudices (for instance, concerning the value of pile-driving formulas, or the interpretation of the results of loading tests) and establishing a more accurate interpretation of actual construction experience. The bulk of the work—the systematic accumulation of empirical data—remains to be done.

Foundation problems, throughout, are of such character that a strictly theoretical mathematical treatment will always be impossible. The only way to handle them efficiently consists in finding out, first, what has happened on preceding jobs of a similar character; next, the kind of soil on which the operations were performed; and, finally, why the operations have led to certain results. By systematically accumulating such knowledge, the empirical data being well defined by the results of adequate soil investigations, foundation engineering could be developed into a semi-empirical science, comparable in its character to certain branches of medicine. For the first time a large-scale effort of this kind was made by the Swedish Geotechnical Committee, in its epochal investigation of Swedish landslides, 1914-22.* In connection with its work, the Committee has tested a vast number of soil samples, extracted from more than 10 000 drill holes, at different depths below the surface. On account of the time (five years) that has elapsed since the Committee finished its work, some of the experimental methods have been superseded. Nevertheless, the work of the Committee could serve as a noteworthy example for the way in which a similar enterprise should be organized in the field of plain foundation engineering.

At present (1927), the data derived from previous experience in foundation engineering are of as little value as were those derived from Swedish landslide experiences prior to the work of the Geotechnical Committee. The information concerning soil character is inadequate, and the interpretation of the observed facts is very often arbitrary and inconsistent with the laws of physics and mechanics. Hence, the first requirement for improving conditions consists in standardizing the methods of soil classification, and, the second, in consistently applying present knowledge of soil mechanics to observations in the field.

* Statens Järnvägars Geotekniska Kommission, 1914–22, Stockholm, May, 1922.

DISCUSSION

CHARLES R. GOW,* M. AM. SOC. C. E.—The theories advanced in explanation of the behavior of soils under various conditions of use open up many new and interesting avenues of approach in the solution of foundation problems involving the predetermining of soil behavior under specific applications of loads. Undoubtedly, from the standpoint of definite and accepted standards of practice, the general status of foundation engineering is at present in the most unsatisfactory condition of any branch of the science with which engineers are called upon to deal.

There is no recognized authoritative source of information available to the foundation engineer by means of which he may act with the reasonable assurance that the requirements of his problem will be properly and efficiently met. Reliance must be had, in general, on observed experience with regard to previous construction undertakings in the same locality, and where such histories are lacking the designer must often proceed on a basis of uncertain assumptions, which may or may not ultimately prove to have been justified.

Heretofore, soils have usually been classified according to their visible physical properties, and engineers have acted on the supposition that these easily recognized characteristics furnished them with a sufficiently satisfactory guide by which to differentiate between soils and identify those possessing similar properties and behaviors.

The results now made available to the profession as a consequence of the author's research investigations throw much new light on the subject. The conclusions thus far would indicate that the essential factors affecting the behavior of soils under pressure are rather more obscure and complex in their nature than has hitherto been generally assumed.

Obviously, such factors as voids ratio, permeability, compressibility, and elasticity, cannot be determined by any superficial examination of soil samples. Intricate and somewhat elaborate laboratory experiments are necessary for securing this information. Yet, assuming the validity of the author's reasoning, these elements must be known before it is possible to predict, with any degree of accuracy, the capacity of a given soil to support a specified load.

The most encouraging feature of this paper is the evidence that the subject of soil mechanics is at last being studied in a scientific manner and that, as a consequence, there are now some positive experimental data which serve to explain many well recognized but heretofore little understood phenomena.

It is known, for example, that sands of similar appearance as to shape and size of grain may behave quite differently under equal conditions of loading. Likewise, certain clays appear to be highly compressible, whereas, other soils of identical appearance will sustain considerable loads without appreciable consolidation. Again, the driving of piles into a given soil may result in a pronounced upheaval at the surface level, while another apparently identical set of conditions produces little or no such action.

It is not suggested that the author's analysis presents a practical method of forecasting these variations in soil behavior; but it indicates the direc-

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tion of inquiry that should be followed in order to discover a satisfactory answer for apparent inconsistencies.

It is true that a determination of the essential factors that would make soil identification positive would probably involve more elaborate and expensive procedure than the ordinary engineer could be expected to command. Nevertheless, once the correctness of the author's theories have been established by a sufficient number of laboratory experiments on soils of known behavior, it should be possible, by means of further research, to calibrate these results against more simple rule-of-thumb tests that could be applied in the field. If a satisfactory method can be developed for accomplishing this result, engineers will have made the greatest single step in foundation engineering that has so far been taken.

The author's analysis of the relation between settlement and foundation area is extremely interesting and more or less in agreement with accepted beliefs; yet it probably will be unwise to assume too readily that such settlements can be reduced to common formulas until at least there has been more practical corroboration of the theory by actual experience.

The author's observations respecting the application of the customary pile-driving formulas are probably justified by the facts, since there are few engineers, having occasion to drive piles under varying conditions, who have not been impressed by the variation between actual and computed capacities. Fortunately, the so-called *Engineering News* formula has usually given results that are at least on the safe side, and until engineers become more enlightened respecting the factors that produce the observed departures from theory, they will probably do well to stick to that formula as the most dependable guide available.

The author does well to emphasize once again the fallacy of relying too strongly on the results of individual pile tests in determining the average pile load of the cluster. His analysis of the distribution of soil stresses, as well as his explanation of the variation between dynamic and static resistance, is especially interesting and enlightening.

Of unusual significance is the author's presentation of the compression and rebound curves (Fig. 14) developed by laboratory experiments on varying types of soil, particularly the comparison presented by similar experiments made on synthetic clays produced by mixing different percentages of powdered mica with fine sand. To those who have been seeking an explanation of the characteristic behavior of clays, as compared with sands, this presentation is especially interesting.

On the whole, it may be said that the author has, by means of this paper, added another important contribution to the clarification of the engineer's understanding respecting many of the most troublesome features of foundation engineering.

E. P. Goodrich,* M. Am. Soc. C. E.—About 1900, the speaker abandoned the use of the *Engineering News* formula. A demonstration of its ineffectiveness was had in connection with some work that was being supervised by a

* Cons. Engr., New York, N. Y.

certain governmental group. Certain requirements were laid down as to the supporting power of the piles, and the *Engineering News* formula was to be used. Those responsible were asked if they would be satisfied with a certain weight of hammer, dropping a certain number of feet. They said they would. The speaker then told them to apply the formula and they found that the result was a negative quantity.

Some data with reference to one of the questions that Professor Terzaghi has propounded, that is, a test of the distribution of stress under a square foundation under which the soil was rather cohesive, are as follows: Some years ago the speaker took a timber platform 4 ft. square, penetrated it by plungers located along a diameter and a diagonal and found that the diagonal stresses were larger than the diametrical ones. In other words, the corners of the square foundation supported more in proportion than any other part, either the center or the sides.

VICTOR A. ENDERSBY,* ASSOC. M. AM. SOC. C. E. (by letter).—The profession should feel indebted to Professor Terzaghi for the clear line he draws between soils to which the *Engineering News* formula is applicable, and those to which it is not. Among field engineers directly in charge of pile-driving, there are a great number who regard this formula as applicable, without modification or question, to every circumstance. There is an equal number who —perhaps more logically—refuse to believe in any formula whatsoever, and who regulate their pile-driving by guess, by the penetration they can force a contractor to attempt, or by the approach of five o'clock. Every practical engineer finds, sooner or later, that there is a great difference in the reliability of formulas in different soils, but there are not many who have any very clear idea as to general distinctions.

It seems very true that the most that can be hoped in regard to the science of foundations is that it become a reasonably reliable empirical system. One is especially impressed with this after such an experience as the writer had on one structure. Certain piles came to refusal in sand at a penetration which he considered unsatisfactory. A jet not being available at the time, he acted on a highly irrational "hunch" and suggested that the piles be left over night and an attempt be made to drive them next day. When this was done, the piles continued to drive with surprising ease, application of the *Engineering News* formula showing that as they stood they had lost 40% of their theoretical bearing power in a few hours. The excavation in which they were being driven was at the time 25 ft. below sea level, and 27 ft. below permanent water line, so that there could have been no question of a change in the texture of material due to subterranean water flows. There was evidently a decompression of some kind, but the occurrence is still a mystery to the writer, which he would like very much to have definitely explained. According to Professor Terzaghi's thesis, this might occur in some soils, but not in sand. The writer has heard a rumor of a similar occurrance on the Gila River in Arizona.

He has had occasion also to test the driving in adjacent and uniform material under three different hammers, a Union No. 2, double-acting, and Vul-

* Constr. Engr. of Bridges, Southern Section, State Div. of Highways, Los Angeles, Calif.

can Nos. 1 and 2, single-acting. The theoretical bearing power for piles of uniform size at uniform penetration with these three hammers was practically the same; the result was a good test for consistency of the formula as applied to various types of hammers.

GEORGE PAASWELL,* M. AM. SOC. C. E. (by letter).—The paper, being synoptical in character, the author, naturally, omitted many vital points which bear on foundation practice as well as on the type of laboratory and field tests which supplied the data for the theory developed. The paper must be a great disappointment to engineers who look for a simple set of rule-of-thumb notes to embrace all foundation practice. It is no mean accomplishment of the author that he has demonstrated the fact that foundation design and construction is as exacting in analysis and as worthy of study as the design of the structures resting on these foundations.

The essential difference between superstructure and sub-surface design is that one may fit the materials to the design of the former; one must fit the design to the materials of the latter. The engineer cannot fit a foundation to a rigid set of formulas. The material must be studied in the site and the foundation must be fitted exactly to it. A code set of bearing values is more than worthless; it is dangerous. The author has illustrated this point very well. Settlement in a foundation has been viewed as an unfortunate and usually unavoidable concomitant of foundation construction; yet such settlement is as inevitable as the deflections and distortions of the supported structure. Provision must be made for them. That makes it necessary to formulate means to determine foundation distortions. The author has briefly indicated how such studies may be made.

The study of soils has indicated that two broad types exist: (1) Those in which the usual stresses of elastic solids are found, modified, of course, by proper elastic coefficients; and (2), those in which hydro-mechanical stresses are set up. Both types can be uniquely identified by sample tests for composition and loading, but neither can be safely described by grain size or moisture contents alone. The author's striking experiment on sand and mica mixtures has demonstrated fully the futility of mere sieve analysis as a basis of soil classification. He has put to rest also the colloid theory as the basis of coherent action in soils.

In his paper entitled "Transmission of Pressure Through Solids and Soils and the Related Engineering Phenomena,"† the writer attempted an analysis of stress distribution through soils and found that certain characteristic failure surfaces existed, similar in type to those found in elastic solids. It was also demonstrated that a uniform distribution of pressure under a load was a fiction of design. Numerous experiments preceding and following the paper have amply demonstrated the truth of this statement.

The huge subway construction program of Greater New York is unquestionably the world's best laboratory for the study and illustration of all types of foundation problems and the writer hopes to see, in the course of the next few years, the crystallization of a definite soil mechanics and the complete abandonment of bearing value codification as a foundation formulation.

* Engr., Corson Constr. Corporation, Brooklyn, N. Y.

† *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 1563.

The author well points out that, given a complete soil identification, there is sufficient mathematical and physical science available to complete the practical investigation, of the foundation. It remains to codify soil identification.

The use of piles has been a blind foundation expedient, save where such piles were driven to absolute refusal. The anticipated capacity of a pile whether determined by formula or by actual field test has been of so indeterminate a character that most foundation failures may be laid to miscalculated pile-bearing values. It has been frequently emphasized by the writer that no adequate pile theory could be formulated until a rational soil mechanics had been developed, for the author has well demonstrated the fact that there is no separate theory for piles and for soil behavior. The development of the pile for underpinning buildings has evolved a better understanding of soil behavior than any other single phenomenon in engineering foundation practice. It is not a question of bearing capacity alone, but of actual load settlement behavior which determines the ultimate safety of the foundation underpinned. The study of the loaded soil and its distortions under the pile toe brought out the reasons for progressive settlement even if a pile had already been tested for a safe load, and there was evolved the ingenious "pre-test" pile, a scientific application of soil mechanics.

The writer is heartily in favor of the abandonment of the *Engineering News* formula for piles, and the adoption of one similar in nature to that suggested by the author, although he would prefer to disregard pile formulas entirely and make a separate study for each foundation type. That would eliminate the dangerous conditions resulting from group application to single pile tests or formulas.

There remains the extremely interesting subject of temporary earthwork supports, such as sheeting and bracing. In this work modern soil mechanics, has been given ample demonstration of its validity. The failure of earth banks is not a simple application of the Rankine or Coulomb special soil-type theory, but is one that can be easily formulated by a study of the elastic properties of the soil. The absolute limits of ground movement are predictable, and that gives a rational method for determining the maximum pressures that may be induced in the sheeting and timbering. This, of course, leads to a far safer and more economical layout of such bracing; but, on the other hand, it requires more study and more design than is usually given to such structures. The use of large excavating machinery, which requires large clearances because of working room, calls for a very careful investigation of load distribution through wales and braces. It must be borne in mind that a movement in the sheeting that permits movement of the earth behind it, also permits the grain re-arrangement of the soil and consequent increase in pressures as is well illustrated by the famous "dilatancy" experiment of Osborne-Reynolds.

C. C. WILLIAMS,* M. AM. SOC. C. E. (by letter).—The principles of soil mechanics presented in this paper should serve as a rational basis for correlating observed facts concerning foundation tests and behavior, and should thus contribute to making the design of foundations more scientific.

* Dean, Coll. of Eng., Univ. of Iowa, Iowa City, Iowa.

That the supporting capacity of soil is not, in general, directly proportional to the area of the foundation, has long been recognized, but the true relationship is not even yet entirely understood. In experiments on alluvial soil made in 1851-52 at New Orleans, La.,* in which areas varying from 0.25 sq. in. to 576 sq. in. were used, the penetration under a constant unit load increased practically with the area. Observations made in France† on alluvial soil indicated similar results. From these and other data,‡ the late Randell Hunt, M. Am. Soc. C. E., drew the conclusion: "Large areas of soft soil will not support as much weight per unit of surface as more limited areas of the same soil." Reflection on the manner of distribution of a local load through an indefinitely extensive solid or semi-solid, would indicate such to be the case.

The relation between the plastic and the elastic deformations of soil should first be recalled. The former is due to displacement of soil particles, whereas the latter is due to the resilient deformation of the particles, without slipping, relative to each other. Contrary to the behavior of steel, the plastic flow precedes the elastic, the latter constituting a small proportion of the total compression. Fig. 17, taken from a test made under the writer's direction by Mr. H. T. Heald, a graduate student, illustrates this relationship for loam and for sand. The experiment consisted of applying and releasing a load which was increased with each application and never entirely removed, the compression and the recovery being recorded at each application.

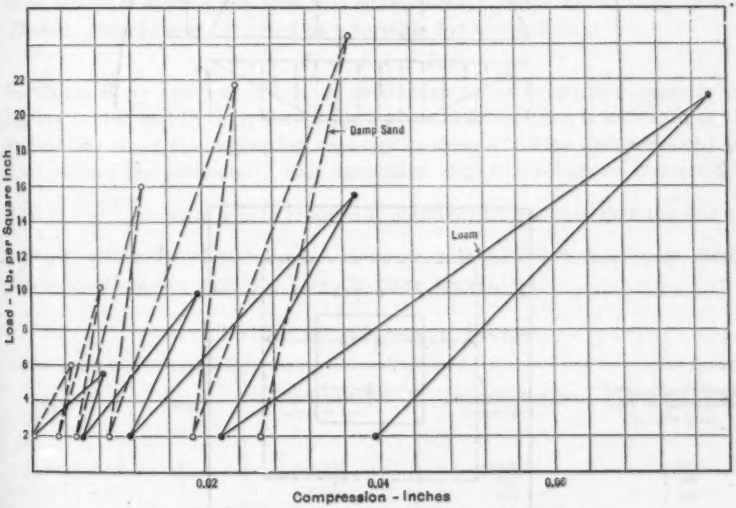


FIG. 17.—COMPARISON OF LOAD COMPRESSION CURVES.

If a load is applied over an area, A, at the surface, the inter-reactions of soil particles along the perimeter of this area are such as to spread the load laterally by virtue of shear over an ever-increasing area of bearing planes at

* *Van Nostrand's Magazine*, Vol. 27.
† *Annales des Ponts et Chaussées*, 1864.
‡ *Proceedings*, Assoc. Eng. Soc., 1888.

successive depths. The rate of this spread depends on the characteristics of the soil, being greatest for coarse fragmental materials, less for granular, and least for silt. Moreover, the spread is only approximately linear and the surface of uniform pressure is curved rather than plane, but the variation from the conception indicated in Fig. 18 is not sufficient to affect seriously the conclusions based thereon. Although the rate of spread is dependent on the soil characteristics, the depth of soil affected, and hence the total spread, depends on the depression of the soil under the load. This deformation increases until the spread distributes the load over an area sufficient to bring the intensity within the elastic resistance of the soil. When a homogeneous soil is assumed, an expression for the total deformation may be obtained as follows:

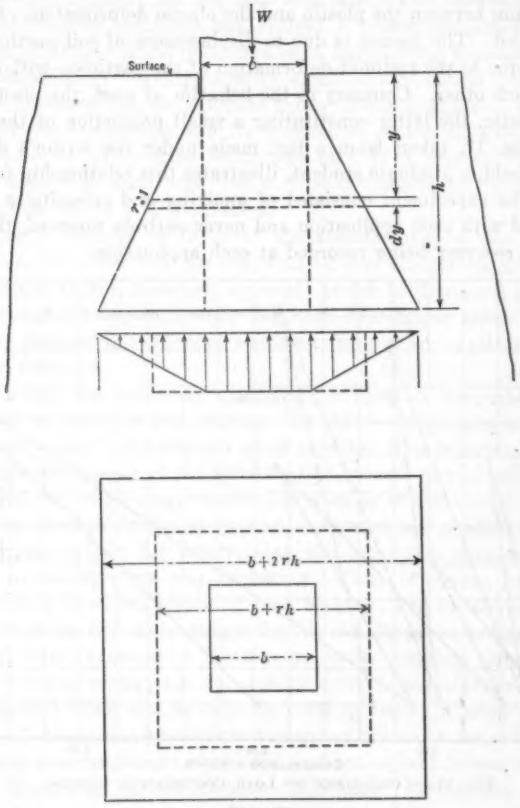


FIG. 18.

Let W be the load applied; h, the total depth of soil compressed; y, the depth to any plane considered; r, the tangent of the angle of spread; and C, the modulus of compressibility. The distribution of pressure on any horizontal plane may be assumed equivalent to a uniform intensity over a width

of $b + r\,y$. Then, for a square surface with side, b, the deformation of an elementary horizontal prism bounded by the sides of the frustum of the pyramid is $\frac{W\,d\,y}{C\,(b + r\,y)^2}$, and the total deformation in a depth, h, is $\frac{W}{C}\int_0^h \frac{d\,y}{(b + r\,y)^2}$, which when integrated gives $\frac{W\,h}{C\,(b^2 + b\,r\,h)}$. This expression shows that the deformation varies inversely with the perimeter, b, as well as with the area, b^2.

If w is the pressure intensity at the surface and p the intensity that the soil will sustain elastically without deformation (at a given depression), then, $w\,b^2 = p\,(b + r\,h)^2$, whence $h = \frac{b}{r}\left(\sqrt{\frac{w}{p}} - 1\right)$, neglecting the weight of the soil itself. If, as for sand under certain conditions, $r = \frac{1}{2}$, and $h = 2$, the deformation would be $\frac{2\,W}{C\,(b^2 + b)}$. In this case the denominator contains simply the sum of the bearing area and the perimeter multiplied by a strip $\frac{1}{4}$ ft. wide.

Thus, an elementary analysis indicates that the supporting capacity of soil depends on two factors: (1) The bearing area; and (2) the perimeter of the bearing surface. However, on account of the variableness of soils, it is impossible to assign values that will be universally applicable to these factors. The relationship may be stated as a formula for convenience,

$$W = A\,p + L\,f$$

in which, W = load; A = area of application; p = supporting capacity per square foot of soil in compression for a given deformation; L = length of the perimeter; and f the supporting capacity in shear (friction and cohesion) per foot around the perimeter. This expression might be otherwise conceived as $\left(A + \frac{L\,h\,r}{2}\right)p$, since p is in reality the intensity of pressure that the soil will sustain within its elastic limit. However, although this form may clarify the conception, the previous form is more convenient.

TABLE 2.—Supporting Factors.

Sand.	Clay.	p, in pounds per square foot.	f, in pounds per linear foot.	Total superimposed load, in pounds per square foot.
0	1	110	400	1 710
1	1	120	280	1 240
5	1	500	160	1 140

These supporting factors, p and f, vary with the character of the soil (mineral and granulometric composition, compactness, water content, etc.), and with the depth to which the load penetrates the soil. The two terms are indeterminate, their relative values depending on which represents the more rigid element in the total support. For the more rigid soils, the direct compressive

resistance becomes the greater, for any considerable compression. Thus, the tests made by A. T. Goldbeck, Assoc. M. Am. Soc. C. E., quoted by the author, give, approximately, the results shown in Table 2 for 0.1-in. penetration and for the various mixes of sand and clay.

At 0.05-in. penetration and less, the marginal shear, in these experiments, carried nearly all the load. Inasmuch as Mr. Goldbeck worked with small compressions, these facts explain the apparent importance of the diameter, $(\sqrt{A})$.

Tests (see Fig. 19) made on a uniform clay soil containing 7% sand in undisturbed condition, using bearing areas of 1.0, 2.25, and 4.0 sq. ft., showed for 0.3-in. depression, $p = 3\,500$ and $f = 1\,500$. For depressions of less than 0.1 in., the perimeter factor carried nearly all the load. In still another test in clay soil, $p = 3\,400$ and $f = 2\,300$ for a settlement of $\frac{3}{8}$ in. While the Austrian experiments quoted by the author are rather fragmentary, the corresponding values at 1.0-in. settlement are approximately, $p = 2\,800$, and $f = 700$.

Of these two elements, p varies almost directly with the settlement in sand and dry clay soils, while, on the other hand, f remains fairly constant, varying, if at all, with some low power of the depth of settlement, in some cases about as the fourth root.

If these elements were determined, the bearing capacity of a larger area could be predicted from a test on a small area. Thus, based on the test on 7% sand soil mentioned previously, using the data from the 0.3-in. penetration, $p = 3\,600$ and $f = 1\,500$, a footing 8 ft. square might be expected to sustain (with 0.5-in. penetration),

$$3\,600 \times \frac{5}{3} \times 64 + 1\,500 \sqrt[4]{\frac{5}{3}} \times 32 = 432\,000 \text{ lb.}$$

or, approximately 5 400 lb. per sq. ft. of superimposed load. The broken-line curves of Fig. 19 show calculated bearings assuming p to vary as the depth, and f as the fourth root.

The correlation between calculated and observed data is high within the ultimate bearing capacity of the soil. The writer has applied this theory to the other test data available, with similar results.

The most reliable practical basis for comparing the supporting capacity of two areas on the same soil, therefore, is the factor, $A + K \times L$, in which, K is a soil coefficient. If, as for a certain sand, K is $\frac{1}{4}$, then a 36-sq. ft. foundation can be expected to sustain on this soil, $\frac{36 + 6}{1 + 1}$, or 21 times the test load on a post 1 ft. square with the same settlement, instead of 36 times as much according to the usual assumption, or instead of 6 times as much, which would be the case if the supporting capacity varied with the diameter for a given penetration.

While for small settlements and for small test areas, observed data, such as those of Mr. Goldbeck's experiments, indicate settlement, under a constant

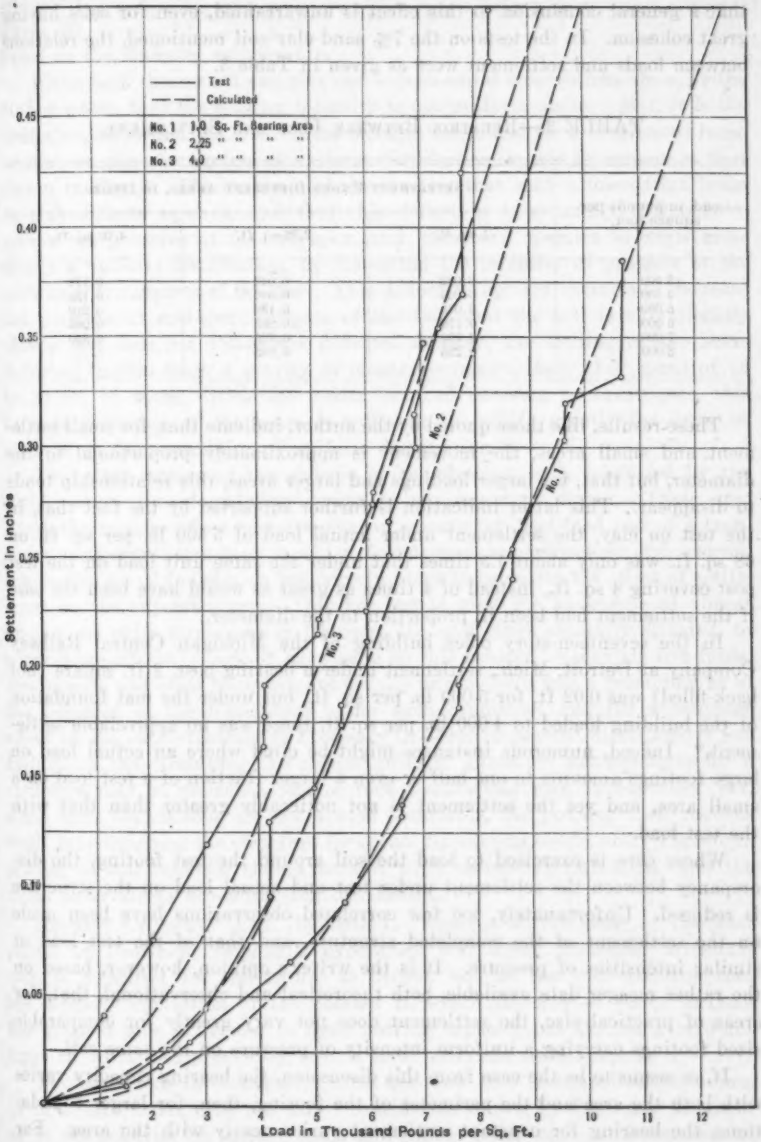

Test
Calculated
No. 1 1.0 Sq. Ft. Bearing Area
No. 2 2.25 " " " "
No. 3 4.0 " " " "
No. 1
No. 2
No. 3
Settlement in Inches
0.45
0.40
0.35
0.30
0.25
0.20
0.15
0.10
0.05
2
3
4
5
6
7
8
9
10
11
12
Load in Thousand Pounds per Sq. Ft.

FIG. 19.

unit loading, to vary with the diameter of the bearing area, the writer believes that a general conclusion to this effect is unwarranted, even for soils having great cohesion. In the tests on the 7% sand-clay soil mentioned, the relations between loads and settlement were as given in Table 3.

TABLE 3.—Relation Between Load and Settlement.

Load, in pounds per square foot.	Settlement Under Different Areas, in Inches.		
	1 sq. ft.	2.25 sq. ft.	4.00 sq. ft.
3 000	0.082	0.042	0.112
4 000	0.054	0.082	0.155
5 000	0.075	0.132	0.210
6 000	0.110	0.205	0.265
7 000	0.152	0.325	0.340
8 000	0.225	0.385	.....

These results, like those quoted by the author, indicate that, for small settlement and small areas, the movement is approximately proportional to the diameter, but that, for larger loadings and larger areas, this relationship tends to disappear. This latter indication is further supported by the fact that, in the test on clay, the settlement under actual load of 5 500 lb. per sq. ft. on 68 sq. ft. was only about 1.5 times that under the same unit load on the test post covering 4 sq. ft., instead of 4 times as great as would have been the case if the settlement had been in proportion to the diameter.

In the seventeen-story office building of the Michigan Central Railway Company at Detroit, Mich., settlement under a bearing post, 2 ft. square (not back-filled) was 0.02 ft. for 5 000 lb. per sq. ft., but under the mat foundation of the building loaded to 4 000 lb. per sq. ft. there was no appreciable settlement.* Indeed, numerous instances might be cited where an actual load on large footings amounts to one-half or even a larger fraction of a test load on a small area, and yet the settlement is not noticeably greater than that with the test load.

Where care is exercised to load the soil around the test footing, the discrepancy between the settlement under test and actual load on the structure is reduced. Unfortunately, too few correlated observations have been made on the settlement of the completed structure, and that of the test load at similar intensities of pressure. It is the writer's opinion, however, based on the rather meager data available, both theoretical and observational, that, for areas of practical size, the settlement does not vary greatly for comparable sized footings carrying a uniform intensity of pressure on the same soil.

If, as seems to be the case from this discussion, the bearing capacity varies with both the area and the perimeter of the footing, then, for larger foundations, the bearing for constant settlement varies nearly with the area. For instance, with two footings on sand, as previously assumed, one 10 ft. square

* *Engineering News*, September 3, and October 1, 1914.

and one 20 ft. square, the respective bearing capacities would be about as $\frac{100 + 10}{400 + 20} = 1 : 3.8$, instead of 1 : 4 when based on the area directly, or 1 : 2 if based on the diameter relationship.

While both theoretical analyses and experimental observations show, as the author states, that the bearing intensity is not uniform under a pier, it is the writer's opinion that the error in the moment calculated for a continuous beam or mat, on the assumption of uniform distribution, is not as serious as that shown in Fig. 3. The author neglects the fact that such a foundation beam or slab deflects upward, and that this deflection is generally sufficient to relieve the pressure at the mid-span, and, therefore, operates to effect more nearly a uniform distribution, by increasing the intensity of pressure at the (downward) supports of the slab. This deflection upward operates to decrease the pressure at mid-span, because of the fact that the soil is only slightly elastic and does not follow the deflected member, the loading of the beam differing therein from a gravity or constant-pressure load. For spans of 16 to 20 ft., or more, which the writer has had occasion to investigate, this upward deflection of the beam is sufficient to nullify greatly the error in moment on the assumption of uniform distribution.

Two other phases of the science of foundations, both suggested by the author, enter into foundation design to an important degree. These are: First, the relative effect on settlement of a constant dead load and of a transitory conjectural live load to which foundations are subjected; and, second, the prediction of foundation behavior in order to calculate the effect of settlement on the superstructure. One has but to visit the basements of almost any of the older structures in Chicago, Ill., and cities similarly situated, to witness the effects of lack of knowledge on this subject. Eccentric live loads, due to wind and other elements, are usually unavoidable, and a prediction as to the behavior of the foundation under such conditions and the consequent effect on the superstructure is necessary in order to prevent distortion, if not failure, of the structure. To secure uniform minimum settlement under a structure may be desirable, just as rock foundations may be desirable; but, since such foundations are not always obtainable, the problem then resolves itself into designing the structure for actual foundation conditions. Therefore, studies of soil behavior should include more detailed information than the usual observations accompanying test loading.

HERBERT CHATLEY,* ESQ. (by letter).—Whether or not some of the finer points of Professor Terzaghi's hypotheses will stand prolonged investigation is a matter for the future to decide, but no one can question that his masterly exposition of the manner in which the flow of water through fine-grained soils is related to their bearing capacity has thrown a brilliant light on what had previously been a most misunderstood subject.

The conditions in Shanghai, China, are those of a saturated soft alluvium of indefinite depth, and the major conclusions drawn by Professor Terzaghi as to the behavior of surface loads and piling under such conditions are of the greatest interest and value to engineers there. Through the initiative

* Acting Engr.-in-Chf., Whangpoo Conservancy Board, Shanghai, China.

of A. V. H. von Heidenstam, M. Am. Soc. C. E., pile tests on rather a large scale have been made for some years in Shanghai by the staff of the Whangpoo Conservancy Board. Professor Terzaghi has been good enough to speak very appreciatively of these tests. Broadly speaking, they have confirmed his views as to piling in soft soils.

The slowness with which fine alluvium expels its contained water is well shown in Shanghai by the behavior of hydraulic fills formed by the pumping up of dredge spoil. Where these fills are above the ground-water level they do become dry, but below the top 2 or 3 ft. some years may elapse before the deeper material becomes fully consolidated. This is true in spite of the fact that the hydraulic filling process tends to remove an appreciable part of the colloid content.

As to the use of piling formulas, the cogency of the reasoning as to the variation of the hydro-dynamic resistance with the duration of the applied force is complete. The new "derivation" of the *Engineering News* formula is most interesting and calls attention to the fact that the assumed constancy of the second term in the denominator of that formula is even more arbitrary than is generally supposed.

In the work on the internal friction of granular masses the author has seen fit to adopt the writer's conclusion* as to the importance of single molecular bonds in uniting particles the weight of which is of the same order as those bonds. The writer has since gone much further into the question, and these studies fully sustain his earlier conclusions as to the magnitude and importance of these forces in granular masses.

In regard to the classification of soils the new data as to the effect of mica are most important. There is one point to which attention might very well be directed and that is the possibilities of change of grain size by the permanent aggregation of particles. Putting it very crudely, one might imagine that, under certain circumstances, a fine clay could gradually be transformed into a loam by the adhesion of the particles.

Engineers would be very glad if the author would give some practical rules for the design of sheet-piling, which acts as a retaining wall.

J. Albert Holmes,† M. Am. Soc. C. E.—Although this paper relates particularly to foundations, it has, together with the author's previous investigations, a bearing on another engineering activity, the investigation, design, and construction of earth dams.

While earth dams of moderate height, built 2 000 years ago and more, are still impounding water and have continued to do so for the greater part of their existence, it is only within the memory of most of the older engineers and within the lifetime of many of the younger ones, that modern engineering thought has been directed toward the problems involved. One of these problems, and the principal one, is the selection and proper placing of materials in a structure. Theory and experience have demonstrated that pressures, tending to disruption, exist in the materials being placed in an earthen dam.

* "On Cohesion", *Philosophical Magazine*, Vol. XL, August, 1920, and "Silt", *Minutes of Proceedings*, Inst. C. E., Vol. CCXII, p. 400.

† Hydr. Engr., Pearse, Greeley & Hansen, Chicago, Ill.

To control this tendency a thorough knowledge of the medium used is as essential as in other engineering works. In investigating and studying the character of materials, permeability or ability to resist the flow of water through them, must be determined.

Several factors enter into the phenomena of flow through soils, as actual size of grain, relative sizes of grain, compactness, and colloidal content. A measure of relative size is indicated by the uniformity coefficient, which means a range of sizes such as will produce a dense mass resulting in small passageways and low percentage of voids. In their natural condition (and it is in this condition that use is made of them), soils have a tremendous range in the size of particles. Writers on the subject of colloids have maintained until quite recently that soils contain not more than 2% of material in a colloidal state. To account for the capacity of soils to absorb dyes and gases, it was assumed that they contained absorptive minerals known as zeolites. Upon investigation no zeolite minerals could be identified in soils. Instead it was found that soils contain from 6 to 70% of colloidal material, in which lies their entire absorptive ability. The inorganic colloidal material in soils is chiefly made up of the products of chemical weathering and decomposition of soil-forming minerals, together with organic matter and perhaps some soil minerals of colloidal size.

An average diameter of 1 micron is arbitrarily fixed by the U. S. Department of Agriculture as the upper limit of colloidal size. At and below this dimension the particles cease to be affected by gravity and are subject to Brownian movement when in suspension in a liquid. Also, at the dividing line of 1 micron between colloidal and non-colloidal particles, microscopical control is good, while, with less diameter, points of emanating light might be mistaken for the particle itself. In solution, those particles that diffuse slowly and, after being deposited, are evaporated and separate into a shapeless jelly, are of one class or group. Those that diffuse rapidly and, under the same process, crystallize, are of a second group. Because of their resemblance to glue the substances of the first group are called colloids, from the Greek word, "Kolla", and for the same reason the second group is called crystalloid. Colloidal material occurs in soils in the gel condition.

Professor W. D. Bancroft, of Cornell University, states that colloidal chemistry is the chemistry of every-day life. It certainly touches engineering frequently and at many points: Pavements; water supply; sewage disposal; foundations; concrete and water-proofing of concrete; earth dams; and agriculture.

The U. S. Department of Agriculture, in its Bureau of Soils* and Bureau of Roads, at Washington, has been investigating the subject of colloids for some time, as has the Society's Special Committee on the Bearing Value of Soils for Foundations, etc. The Department of Agriculture is interested in the subject because it bears on the fertility of soils. The retention of moisture and the chemical action of fertilizers are dependent on the quantity and character of colloidal matter present in soils. The colloidal condition of soils enters very materially into the problem of dam construction.

* *Bulletins 1122, 1193, 1311, 1452*, U. S. Dept. of Agriculture.

One of the recent accomplishments of the U. S. Department of Agriculture in its Bureau of Soils was to determine the chemical composition of forty-five samples of soils and subsoils, collected from widely distributed points in the United States and representing many types. Peats and mucks were not included. The colloidal material was extracted from these samples and the chemical composition of both soils and colloids determined. It was found that soils and extracted colloids alike contained the following simple chemical compounds and elements, but in varying quantities:

Silica	SiO_2	Soda	Na_2O
Titanium	TiO_2	Phosphoric acid	P_2O_5
Alumina	Al_2O_3	Sulfur	SO_3
Ferric oxide	Fe_2O_3	Chlorine	Cl
Manganese	Mn O	Water	H_2O
Lime	Ca O	Nitrogen	N
Magnesia	Mg O	Organic matter	....
Potash	K_2O		

Carbon dioxide, CO_2, was obtained from three soil samples and one sample of colloid.

Silica is the principal constituent of soils, and it was found that the colloidal matter is composed mainly of silica, alumina, iron oxide, and combined water, the percentage varying in the samples as follows:

	In Soils.	In Extracted Colloids.
Silica	51.32 to 93.66	31.84 to 55.44
Alumina	2.57 to 22.92	16.42 to 38.28
Iron oxide	0.93 to 13.82	4.66 to 16.67
Combined water	.....	3.33 to 16.56

The colloids in the soils ranged from 6.2 to 57.8 per cent.

The colloidal material differs chemically from the larger soil particles, but separate and distinct compounds were not identified. The colloidal mixtures are very intimate and are without tendency to break up into known compounds. Table 4 illustrates the differences in composition of colloidal matter and whole soil and the coarser particles of soil.

TABLE 4.—AVERAGE COMPOSITIONS OF SOILS, COLLOIDS, AND COARSER MINERALS COMPARED AS PERCENTAGES.

	SiO_2.	Al_2O_3.	Fe_2O_3.	CaO.	MgO.	K_2O.	Na_2O.	Combined H_2O.
Colloidal matter from 45 samples	43.34	26.84	10.41	1.04	1.72	1.43	0.38	9.93
Coarser mineral particles from 35 of the 45 samples	87.2	6.0	1.9	0.5	0.5	1.5	0.9	0.6
Mineral particles in fine sands and silt of a different series of 26 soils	83.3	7.3	1.5	2.0	1.7	2.9	0.9	0.4

The nature of soil colloids is destroyed by heat. Two samples, one of colloidal or ultra-clay, the other of clay soil, made into a number of test

pellets, were subjected to carefully controlled heat. At desired temperatures a part was removed and the temperature was stepped up on the remainder. As the pellets were removed from the furnace, ammonia absorption determinations were made, with the following results:

Colloid Sample:

Temperature, in degrees centigrade	110	265	374	559	754	1130
Cubic centimeter of NH_3 absorbed per cubic centimeter of colloid	110.3	100.8	80.0	74.1	57.5	2.2

Soil Sample:

Temperature, in degrees centigrade	110	190	265	374	522	673	844	1130
Cubic centimeter of NH_3 absorbed per cubic centimeter of soil	27.7	25.3	24.8	19.7	14.9	13.6	7.4	1.4

The colloids were progressively destroyed, and under the high temperatures quite completely destroyed.

Extracting the colloids from a soil to determine the amount of colloidal material is a long and laborious process. A more rapid method, checked by the extraction process, is the use of dyes, gases, and water vapor.

It was found that practically all the colloids in a soil are associated with the finer particles, and very few or none with the coarser parts. Certain ratios were found between component parts of the colloids which gave a correlation. This correlation, associated with rainfall, temperature, and color, has been presented as an index of the soils for agricultural purposes.

The speaker studied the data presented by the experiments with the object of finding a relation that might be used in the selection of soils for dam building. The most satisfactory is that existing between the colloidal content and loss by ignition. The process of incinerating samples of soil to determine their "organic" content is not a new idea, but a knowledge of the colloidal content, carefully determined, of a series of soil samples that have been incinerated, is new.

The relation between colloidal content and ignition loss is shown in Fig. 20, which indicates the determinations from thirty-five samples. The diagram is offered as a means of determining approximately the amount of colloidal material in soils proposed to be used in dam construction. The incineration of a soil can be accurately and quickly done. The extraction of colloidal material from the thirty-five soil samples was almost complete. It may be seen by the diagram that practically all the samples range between 2 to 8% ignition loss and 5 to 35% colloidal content.

The process of sluicing earth by the semi-hydraulic or the hydraulic method, would probably separate less than one-half the amount of colloid that is removed from a soil by laboratory methods; that is, an ignition loss of 5% would not mean that 25% of the colloids could be separated by sluicing, although approximately that amount is in the soil. All of it would be

sluiced, with the finer particles, into the core where, by its gel-like condition and by swelling, it would fill the voids between the particles. It is as undesirable to have too little colloidal material in a dam as too much. Both these extremes should be avoided by the selection of a material containing the right amount of colloids. The colloidal content of core material being placed in a dam may be ascertained by incineration with the assurance that the result is reasonably correct.

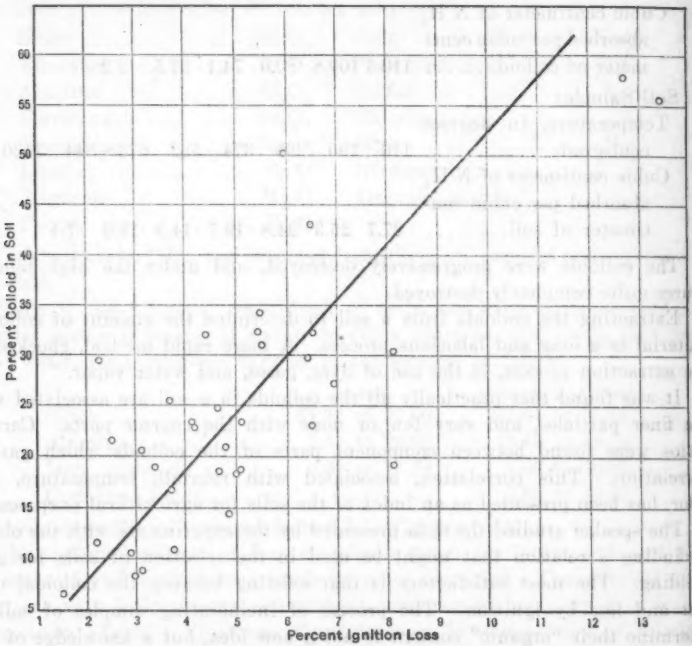


FIG. 20.—RELATION BETWEEN IGNITION LOSS AND COLLOIDAL CONTENT OF SOILS.

No permeability tests were made by the U. S. Department of Agriculture on the soils investigated for colloidal content and chemical composition. The speaker has compiled data, from various sources, on the relation between ignition loss and permeability. These data are given in Table 5, in Columns (5) and (6) of which it may be seen that the percentage of ignition loss increases in reasonable regular order as the permeability rate decreases.

Having this knowledge and having determined the colloidal content of a soil as indicated by Fig. 20, or by a more precise method, a solution of the problem of selecting a proper material for an earth dam is made more possible. Effective size and the uniformity coefficient of a fine-grained soil have no relation to its permeability rate, although a soil having a high uniformity coefficient insures small void spaces and low rates of flow.

A gramme of colloid, containing particles having an average diameter of 91 millimicrons, has a surface area of particles amounting to 24.2 sq. m. Be-

cause of this enormous area the surface forces developed are sufficient to make the colloid a stronger binding agent than Portland cement. This is true only when the material is dry. The experiment cited was not carried beyond a 10% content of the binding agent, although it was sufficient to show that colloidal material is the principal binding agent of soils, giving them cohesiveness according to the moisture content.

TABLE 5.—DATA SHOWING THE RELATION BETWEEN THE COLLOIDAL CONTENT (IGNITION LOSS), AND MAXIMUM RATE OF SEEPAGE THROUGH SOILS.

No. (1)	Material. (2)	Effective size,* in millimeters. (3)	Uniformity coefficient.† (4)	Ignition loss, percentage. (5)	Maximum rate.‡ (6)
1a	Very fine sand	.....	.....	0.00	514 000
1b	Very fine sand	.....	.....	0.00	387 000
2	Very fine sand	0.023	2.4	1.36	375 000
3	Very fine sand	0.02	2.0	1.54	256 100
4	Upland soil	0.054	5.1	4.37	26 840
5	Subsoil	.....	.....	3.32	21 350
6	Fine soil, sandy, roots	0.026	9.3	4.52	21 200
7a	Top-soil and subsoil	.....	.....	3.67	15 300
8	Sandy soil	0.091	2.9	3.97	15 000
9	Fine sand	.....	.....	4.12	11 700
10	Top-soil, roots in	.....	.....	5.74	10 800
11	Top-soil, roots out	.....	.....	4.80	10 000
7b	Top-soil and subsoil	.....	.....	3.67	8 600
12	Gravelly subsoil	.....	.....	3.01	6 300
13	Fine sandy soil	0.04	5.9	6.79	3 550
14	Subsoil and sand	.....	.....	3.46	3 200
15	Top-soil and subsoil	.....	.....	5.21	2 200
16	Top-soil and subsoil	.....	.....	7.00	1 500

* "Effective size" is the size of grain in materials than which one-tenth of the sample is finer and nine-tenths coarser. The finer one-tenth controls the seepage rate.

† "Uniformity coefficient" is the relation, expressed as a ratio, between the grain size which has 60% of the sample finer than itself and that which has 10% finer than itself.

‡ "Maximum rate" is the quantity of water, in gallons per acre per day, that will seep through a material, with the loss of head equal to the depth of the material.

In a homogeneous structure, either compacted in layers or consolidated by irrigation, the function of water is to soften the colloids in the dry, hard soil, breaking it up and permitting its particles to re-arrange themselves in a more compact form and by the continued presence of water, in order to keep the colloidal material expanded in the voids of the soil. Colloids have capacities for swelling of from 40 to 150% and a water-holding capacity of 50 to 140 per cent.

Investigation of the permeability rate, combined with a determination of colloidal content and ignition loss, is very much needed. In making a mechanical analysis to determine the uniformity coefficient the "clay" fraction, diameter 0.005 to 0 mm., roughly checks the colloidal content.

The speaker offers the foregoing for consideration and suggests a line of research in soils, physical and chemical as well as mechanical, that will supplement the data given.

JACOB FELD,* ASSOC. M. AM. SOC. C. E. (by letter).—In 1727, Couplet published his monumental work on soil pressures, giving a "complete theoretical analysis of soil physics," and, now, 200 years later, Professor Terzaghi

* Cons. Engr., New York, N. Y.

writes that the science of foundations is of the present and future. From this it must be assumed, that the science of foundations has no past, and there is no possibility of criticizing that statement if the rigid definition of a science is insisted upon. Still, it is difficult to slight such announcements as appear in the *Annales* of the French Academy of Sciences (January 22, 1783) to the effect that the problem of determining the lateral earth pressure was susceptible of a rigorous solution for any special case by the application of the theory outlined on that date by Chauvelot.

The design of foundations has never been based on a true science of foundations, and, to disagree with the author, it is not even now (in 1928) based on any scientific principles. There are, in common use, certain methods on which various types of foundation design are based. Such methods are adaptations of principles of mechanics, taken either from statics or pneumatics, with or without correction factors.

It cannot be expected that the old, although not accurate, methods of design should be discarded in favor of principles from a "science of foundations" when such science does not exist in the common knowledge of the day. As the author states, there is insufficient data available from which even empirical rules may be deduced, and so little is known of the materials which are encountered (soils and rocks) that no skeleton or shelving of general data and hypothesis can be formed as a basis for the accumulation of missing information. Until such a framework can be formed, no one can announce even the birth of a science of foundations.

The present methods for the design of foundations are open to many more criticisms than are enumerated by the author. However, these methods have been used for the preparation of plans for many foundations which, for the large percentage of cases, have been successful. Note that this statement uses the expression "preparation of plans" and not "design". The success of a plan is measured by the result obtained in supporting a superstructure without apparent or measurable danger. The economy of a foundation plan is quite another matter.

It is quite difficult to clear away all this accumulation of precedent and "experience" and suddenly substitute a new "science of foundations." Certainly, it is not a one-man job and no one-man's work can be expected to supply all the necessary frame upon which such a science can be built. The profession should be very thankful to the author for his numerous reports of the past few years, all pointing to the desired goal. It is high time that the Society has produced some constructive results in providing a basis for the development of this much needed science.

The greater part of the work in the subject of soils during the last three centuries has been in the development of the art of foundation construction rather than in the preparation of data for the scientific development of foundation design. The method of shoring or underpinning a structure is far more advanced than the design of the foundation which may be placed under such structure. The chief drawback to the lack of scientific development has been the absence of any authoritative nomenclature or definite stand-

ardization of terms. The tendency has been to copy expressions from the mechanics of solids, although it has been many years since soil mechanics was separated from hydrodynamics and from the statics of solids. Some writers have gone far in the opposite direction, coining new expressions which, often after translation, are crude and misinterpreted or entirely incomprehensible to the reader.

The relation between the size and carrying capacity of a loaded area can never be rigidly fixed by any usable expression. The author deals only with the individual case, corresponding to the case of a single pile, and has omitted entirely the discussion of the effect of loaded areas in close proximity on each other. Even in the individual case, the author has omitted a very important factor, namely, the shape of the loaded area. In the bearing of a solid against a solid, within the elastic limits, the usual assumption that the shape of the loaded area has no effect, probably brings in no appreciable error. The study of loaded plates of various shapes leads one to doubt the accuracy of the assumption. In a perfectly elastic material, with no cohesion and no surface tension, the shape of a loaded area can have no effect. However, in a substance like a clay, there is a resistance to settlement under loads (bearing power, so-called), which is a function of the area and also a function of the perimeter. The latter term is now being disregarded. Some of the author's conclusions should be modified to suit this perimetral resistance. Two circular areas of diameters in the ratio 1:2, loaded equally per unit area, have equal resultant resistances as a function of the area, but the smaller has twice the length of edge per unit area. The smaller area will settle the least.

An equally important problem is the determination of the effect of one loaded area upon adjacent areas. The complexity of such a usual problem as the design of spread footings for the columns of a frame building with its interior and exterior footings as well as square and non-square loaded areas, is appalling when one thinks of the speed with which plans for such footings are being prepared daily. Many instances are known where unequal settlement is very evident. Unfortunately, few such cases are on record.

Still another disregarded factor is the amount and nature of cover (backfill) above the bearing area of a footing. In a few isolated ideal cases, theoretical methods seem to point to the true values of the side frictional support of piers or caissons, based on the formulas of Coulomb and others, centuries old.*

Edge resistance and side-frictional resistance tend to equalize the distribution of resulting carrying capacity per unit base area. The true state of affairs is much closer to the uniform than to the parabolic distribution of the soil reaction. The diagrams in Fig. 3 are an indirect proof of the last statement; for, if the parabolic distribution results in bending moments in continuous footings of more than twice the calculated moment (based on a uniform distribution), it is inconceivable why at least one-half such footings built have not failed. The "factor of safety" of a structure, based on good

* *Engineering News-Record*, Vol. 88, 1922, p. 1052.

concrete and a reduced load-summation design and built with concrete "not quite as good as specified", is certainly in the neighborhood of two.

The writer would like to bring up a problem which has interested him in connection with a recent arbitration in which he was involved. If one of two exactly similar six-story buildings is removed, and replaced by a taller building with footings placed at exactly the same depth as had previously existed, but with greater load intensity, should there be expected a settlement of the adjacent building, which has not been disturbed? Is such a design proper, or should some means be devised to prevent the old building from settling, even if the new footings are not built to any level lower than the sub-grade of the adjoining building? The problem has also some interesting legal aspects. The writer might mention his argument that, even if the new footings are built no lower than the sub-grade of the old ones, their necessary settlement on receiving load will cause them to take a position sufficiently lower to cause a "sympathetic" settlement of the adjoining footings.

The subject of piles for foundation purposes is very ably treated by the author. The objection to the use of the recognized pile-driving formula is a special case of the general objection made by the Belgian scientist, Boussinesq, that all soil pressure designs are fundamentally problems in statics and the experimental work on which such designs are based, is dynamic in method.

As to the basis for a soil classification, the writer* is still of the opinion that:

"* * * the classification of soils and other granular materials should be along the lines of strain characteristics rather than those of stress, namely, elastic strain, plasticity, and fluidity, for these phenomena are evident and measurable, and embrace within them the results of all the stresses which may act. Generally speaking, the elasticity factor will take care of the solid, the fluidity of the fluid, and the plasticity of the colloid properties of the material being classified."

W. S. HOUSEL,† ASSOC. M. AM. SOC. C. E. (by letter).—Without doubt, it is a source of satisfaction to many to see a revival of interest in the problem of the bearing value of soils. A number of men, including the author, have devoted a great deal of time and effort to a better understanding of foundation problems from the standpoint of soil conditions.

During 1927, the writer, acting for the Board of Wayne County Road Commissioners at Detroit, Mich., has been engaged in conducting a most extensive series of tests to determine the bearing value of soils. The work has been done in the field, under conditions governing actual construction, and, at the same time, every effort has been made to keep the accuracy of the data comparable to what might be obtained with a similar set-up in the laboratory.

More than fifty tests have been made on different sizes and shapes of bearing areas, some tests on piles, and some on combinations of bearing plates and piles. Measurements of lateral pressure and a study of the general phenomena of the transmission of pressure in soils constitute an important part of the

* *Transactions*, Am. Soc. C. E., Vol. LXXXVI (1923), p. 1567.
† Instr. in Civ. Eng., Univ. of Michigan, Ann Arbor, Mich.

investigation. The laboratory work is being conducted by the Civil Engineering Department of the University of Michigan. Through the courtesy of the Michigan State Highway Department, the facilities of the State Highway Laboratory have been made available for this work. With the field results of this investigation at hand a few comments on the principles of soil action as outlined by Professor Terzaghi may be of interest.

The most consistent results of all the tests made by the writer indicate that there are two factors of strength by virtue of which cohesive soils, such as clay, are capable of supporting loads. One of these factors is the shearing resistance on the perimeter of the bearing area; the other is the resistance of the soil to compression, due to a condition of strain set up in the soil around the bearing area. This last phenomenon has been called a pressure bulb.

During the progress of a test it is noticeable that the first failure taking place is failure in shear along the perimeter of the bearing area. In the latter part of the test there is a noticeable upheaval of soil surrounding the bearing area which indicates that a condition of strain exists. That this upheaval cannot be classified merely as flow of soil from beneath the bearing plate is shown by the fact that for some time after the upheaval is first noted the bearing plate continues to support the increased load applied with relatively small settlements. This action continues with increased loads up to a rather definite point of failure, at which point the soil flows from beneath the bearing plate with apparently no restraint. This marks the point at which the pressure bulb fails.

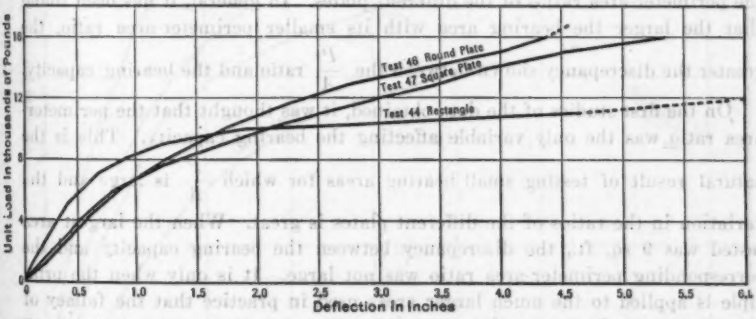


FIG. 21.—COMPARISON OF DIFFERENT SHAPED PLATES OF SAME AREA, 4 SQUARE FEET.

The load deflection diagrams obtained by the different tests bear out these general observations. Fig. 21 shows a set of typical load deflection diagrams, in this case for three different bearing plates, each with an area of 4 sq. ft. As shown, one plate was round, one square, and the third rectangular, with a ratio of width to length of 1 to 7. The round plate has a perimeter-area ratio, $\frac{P}{A}$, of 1.78; the square plate, of 2.0; and the rectangular plate, of 3.04. The material is stiff yellow clay and the size of the test pit is the same as the bearing plate, 4 sq. ft.

It is significant that, in the first part of the tests, the range of bearing capacity in which shear failure about the perimeter is taking place, the relation between the bearing capacity of the several plates is similar to the relation between the values of x for the different plates. The bearing capacities are not in exact proportion to x because, even in their lower ranges, the pressure bulb is beginning to have some effect as a factor of strength.

In the latter part of the test, in the range of bearing capacities where the pressure bulb is a controlling factor, there is a complete reversal of bearing capacity relations. The plates with the largest $\frac{P}{A}$ values have the smallest bearing capacities. The important consideration in upper range of capacities is the effectiveness with which the different shaped plates adjust themselves to the pressure bulb. The strained condition in the soil, giving rise to the phenomenon of the pressure bulb, tends to become circular. Thus, the round plate is the best shape to take advantage of this factor of strength, the square plate is not quite so good, and the rectangular plate is the least effective of all.

Fig. 22 shows the load deflection diagrams obtained by testing areas of 1, 2, 4, and 9 sq. ft. in the same shape of plate. As in Fig. 21, the material is yellow clay and the pit is the same shape and size as the bearing plate in each case. The decreased capacity of the larger plates with smaller values of $\frac{P}{A}$ is the first impression gained from a study of these diagrams. Again, it is significant that the variation in bearing capacity is not in direct proportion to the perimeter-area ratios of the different plates. In general, it has been found that the larger the bearing area with its smaller perimeter-area ratio, the greater the discrepancy shown between the $\frac{P}{A}$ ratio and the bearing capacity.

On the first studies of the data obtained, it was thought that the perimeter-area ratio was the only variable affecting the bearing capacity. This is the natural result of testing small bearing areas for which $\frac{P}{A}$ is large and the variation in the ratios of the different plates is great. When the largest area tested was 9 sq. ft., the discrepancy between the bearing capacity and the corresponding perimeter-area ratio was not large. It is only when the principle is applied to the much larger areas used in practice that the fallacy of this assumption becomes apparent.

However, for the purpose of discussion, it may be profitable to start with this assumption, which is admitted to be false, and develop the principle as applied to the action of soils and see where it leads. It is assumed that the bearing capacity of two spread foundations of different sizes is proportional to their perimeter-area ratios, the settlement being constant in both cases.

Let p be the bearing capacity, P, the perimeter, and A, the area of the footings considered. Then, for a constant settlement in the two footings, and according to the assumption,

$$\frac{p_1}{p_2} = \frac{P_1}{A_1} \div \frac{P_2}{A_2} \quad \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots (7)$$

If the bearing areas considered are round: $P_1 = \pi D_1$; $P_2 = \pi D_2$; $A_1 = \frac{\pi D_1^2}{4}$; and $A_2 = \frac{\pi D_2^2}{4}$. Substituting these values in Equation (7) and simplifying:

$$\frac{p_1}{p_2} = \frac{D_2}{D_1} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (8)$$

If the bearing areas are square: $P_1 = 4 D_1$; $P_2 = 4 D_2$; $A_1 = D_1^2$; and $A_2 = D_2^2$. Again, substituting in Equation (7) and simplifying:

$$\frac{p_1}{p_2} = \frac{D_2}{D_1} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (9)$$

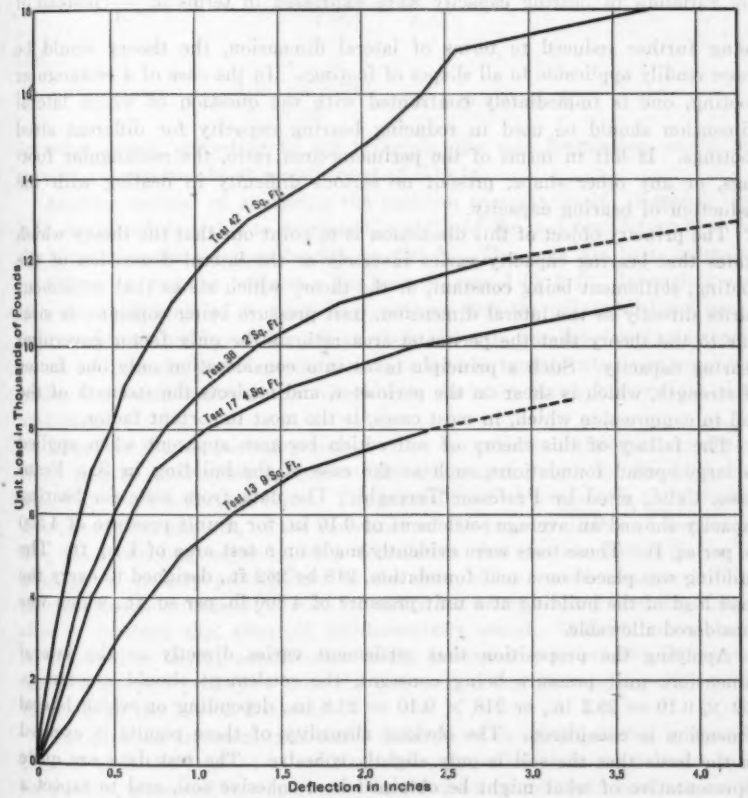


FIG. 22.—COMPARISON OF ROUND PLATES WITH DIFFERENT AREAS.

Thus, it would seem that bearing capacity varies inversely as the diameter in round plates, and inversely as the lateral dimension in the square plates, when the settlement is constant. If the bearing capacity or unit pressure were kept constant, the settlement would vary directly as the lateral dimension, being of course greater for the larger bearing areas. This conclusion,

based on an assumption which is believed to be false, corresponds exactly to the theory as stated by Professor Terzaghi. As has been shown, this mistaken theory is the natural result of insufficient data on small sizes of test areas.

Aside from the consideration that the theory is based on a false assumption, there seems to be additional objection to the use of such data as have been obtained, in terms of settlement for different sizes of foundations. To state the theory in terms of bearing capacity for a given settlement seems to be more direct and better adapted to application to practical problems than to leave it in terms of settlement for a given unit pressure. In addition, if the variation in bearing capacity were expressed in terms of $\frac{P}{A}$, instead of being further reduced to terms of lateral dimension, the theory would be more readily applicable to all shapes of footings. In the case of a rectangular footing, one is immediately confronted with the question of which lateral dimension should be used in reducing bearing capacity for different sized footings. If left in terms of the perimeter-area ratio, the rectangular footings, or any other shape, present no serious difficulty in dealing with the reduction of bearing capacity.

The primary object of this discussion is to point out that the theory which states that bearing capacity varies inversely as the lateral dimension of the footing, settlement being constant, or the theory which states that settlement varies directly as the lateral dimension, unit pressure being constant, is similar to the theory that the perimeter-area ratio is the only factor governing bearing capacity. Such a principle takes into consideration only one factor of strength, which is shear on the perimeter, and neglects the strength of the soil in compression which, in most cases, is the most important factor.

The fallacy of this theory of soil action becomes apparent when applied to large spread foundations, such as the case of the building in San Francisco, Calif., cited by Professor Terzaghi. The data from tests for bearing capacity showed an average settlement of 0.10 in. for a unit pressure of 4 800 lb. per sq. ft. These tests were evidently made on a test area of 1 sq. ft. The building was placed on a mat foundation, 218 by 252 ft., designed to carry the dead load of the building at a unit pressure of 4 800 lb. per sq. ft., which was considered allowable.

Applying the proposition that settlement varies directly as the lateral dimension, unit pressure being constant, the settlement should amount to $252 \times 0.10 = 25.2$ in., or $218 \times 0.10 = 21.8$ in., depending on which lateral dimension is considered. The obvious absurdity of these results is excused on the basis that the soil is only slightly cohesive. The test data are quite representative of what might be obtained in a cohesive soil, and to expect a settlement under such conditions comparable to the results obtained from the equations given, would be no less absurd, even if the soil were of the most cohesive type.

To analyze the test data obtained in the example under consideration in terms of bearing capacity instead of settlement leads to an interesting result. Suppose that it was decided to set 0.10 in. as the limiting settlement, and it

was desired to find the bearing capacity of a mat foundation, 218 by 252 ft., which would not exceed this limiting settlement. It is assumed that the tests were made on a round plate (1 sq. ft.) with $\frac{P}{A}$ equal to 3.55. The perimeter-area ratio of the mat foundation is,

$$\frac{(218 \times 2) + (252 \times 2)}{218 \times 252} = 0.017$$

Let p be the bearing capacity of the large foundation. Using the direct ratio of bearing capacities to perimeter-area ratios:

$$\frac{p}{4\,800} = \frac{0.017}{3.55}$$

and,

$$p = \frac{0.017 \times 4\,800}{3.55} = 23.0 \text{ lb. per sq. ft.}$$

In other words, according to the principle, the mat foundation would not support its own weight at a settlement of 0.10 in.

Another method of analyzing the problem presents itself. Suppose that it is considered desirable to increase the width of the mat foundation by 1 ft., the dimensions would then be 219 by 252 ft. The area has been increased by 252 sq. ft. and the perimeter has been increased by 2 ft. In such a case it would be difficult to convince any engineer that the only additional carrying capacity gained by adding 252 sq. ft. to the area was the shearing strength on an additional 2 ft. of perimeter. Yet that is exactly what the principles under discussion mean, inasmuch as x is the only factor involved in their application.

In spread foundations of practical sizes, the perimeter-area ratio is always small and in such cases to neglect the shearing strength on the perimeter would not lead to serious error. The other factor of strength is the compressive strength of the soil, or the resistance of the pressure bulb to deformation. The total carrying capacity of a foundation due to this second factor of strength is directly proportional to the area. This would seem to indicate that the much criticized practice of using the same bearing capacity for all sizes of footings was, after all, fundamentally sound. This statement could be defended as having considerable merit, inasmuch as it involves less departure from sound principles than the proposition that bearing capacity varies inversely as the lateral dimension.

The greatest criticism that could be made of existing practice in foundations is that the selected allowable bearing capacity is in most cases based on a guess, governed by experiences in previous foundations, and not on the most reliable information. It seems fair to make the statement that, at present, the only reliable method of determining bearing capacity is by actual test.

Inasmuch as it is not feasible to test areas as large as the practical footing, the tests on smaller areas seem to be the only solution. The small areas involve a large perimeter-area ratio and the shearing strength on the perimeter

is a very considerable factor of strength. It thus becomes necessary in reducing the bearing capacity to practical sizes, to consider this factor, but not to the exclusion of the other and more important factor of strength due to the resistance of the soil to compression.

The equation for bearing capacity that the writer has found most useful in interpreting test data, and which fits the test data most satisfactorily, includes both factors of strength.

Let p = bearing capacity, in pounds per square foot.
m = shear on the perimeter, in pounds per linear foot.
n = resistance of the soil to compression, in pounds per square foot.
P = perimeter of bearing area, in feet.
A = area of footing, in square feet.
W = total allowable load on the footing.

Then,

$$W = A\,p = P\,m + A\,n$$

and,

$$p = \frac{P\,m}{A} + n \qquad (10)$$

Let the perimeter-area ratio, $\frac{P}{A} = x$; then,

$$p = m\,x + n \qquad (11)$$

A study of the form of Equation (11) reveals that it is a linear function involving two variables, namely, bearing capacity and perimeter-area ratio. If the bearing capacity for a certain type of soil is determined by test for two bearing areas having different perimeter-area ratios, it is possible to solve the two resulting equations for the constants, m and n. Having thus determined the shearing strength on the perimeter and the compressive strength of the soil for which the tests were made, the bearing-capacity equation (Equation (11)) can be used to determine the allowable unit pressure for larger footings such that the limiting settlement will not be exceeded.

In Fig. 23 bearing capacity has been plotted against perimeter-area ratio and the corresponding equations for these curves are noted in Table 6. Curves 1, 2, 3, and 4 are bearing capacities for a 1-in. deflection, which was taken as the allowable settlement. Curves 5 to 10 are for the bearing-capacity limit, which is defined as that unit pressure beyond which progressive settlement takes place. It is the unit pressure beyond which the material acts as a viscous fluid, and the consolidation of the material is not sufficient to permit a condition of equilibrium to be attained with a comparatively small amount of settlement.

These equations and curves have been determined by series of tests on the three different types of soil indicated as yellow clay, stiff yellow clay, and blue clay. This classification is merely used as identification and is not intended to have any other connection with the physical characteristics of the material. These characteristics are given by Figs. 21 and 22.

There are a few points worthy of emphasis in connection with this set of bearing-capacity equations and curves (Fig. 23):

(1) As the size of the footing is increased and x approaches zero the bearing capacity approaches, not zero, but a constant value which is the compressive strength of the soil, and is indicated as the intercept on the load axis.

(2) For a type of soil which has a high degree of fluidity and low cohesive strength the bearing capacity is more nearly constant for all sizes, indicating that x is the least important factor in the bearing capacity.

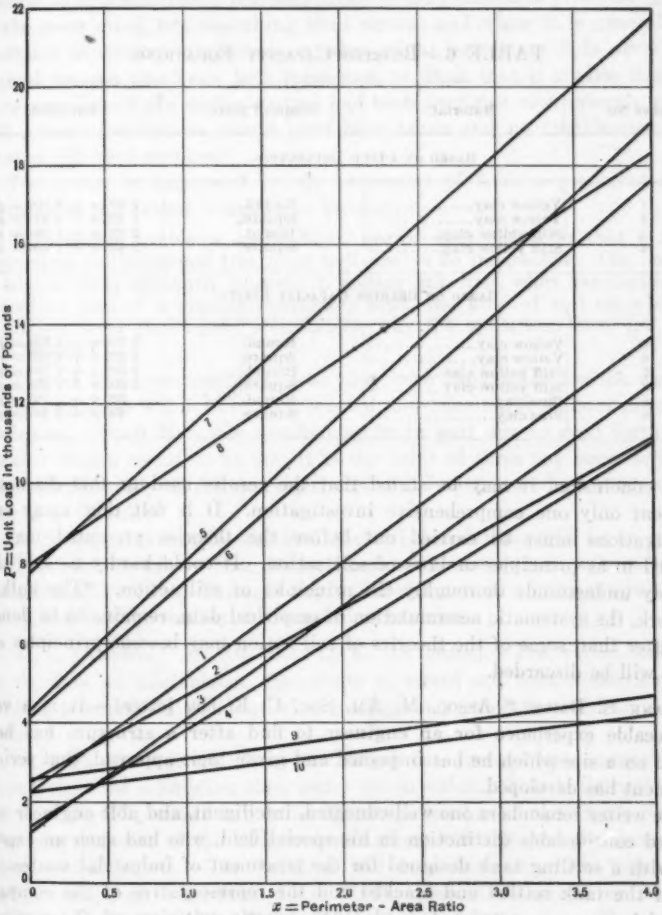


FIG. 23.—BEARING CAPACITY CURVES.

(3) In practical sizes of footings, having perimeter-area ratios between 0 and 1, the shape of the footing, whether round or square, is not so important. This is consistent with the idea that the loss in bearing capacity (as controlled by the pressure bulb), due to the sharp corners, is a smaller part of the total

bearing capacity in large footings. In the smaller areas used in the tests, the loss due to sharp corners is large in proportion to the total area of the plates.

(4) In a type of soil, such as the blue clay, having a high degree of fluidity, the pressure bulb is a controlling factor throughout, and the difference in bearing capacity, due to the shape of the plate, is practically constant. This is clearly shown by the fact that Curves 9 and 10 (Fig. 23) are almost parallel.

TABLE 6.—BEARING CAPACITY EQUATIONS.

Curve No.	Material.	Shape of plate.	Equation.
	BASED ON 1-INCH DEFLECTION.		
1	Yellow clay	Round.	$2\,880\,x + 2\,260 = p$
2	Yellow clay	Square.	$2\,150\,x + 2\,530 = p$
3	Stiff yellow clay	Round.	$2\,820\,x + 1\,260 = p$
4	Stiff yellow clay	Square.	$2\,420\,x + 1\,420 = p$
	BASED ON BEARING CAPACITY LIMIT.		
5	Yellow clay	Round.	$3\,700\,x + 4\,300 = p$
6	Yellow clay	Square.	$3\,370\,x + 4\,170 = p$
7	Stiff yellow clay	Round.	$3\,500\,x + 7\,780 = p$
8	Stiff yellow clay	Square.	$2\,600\,x + 7\,950 = p$
9	Blue clay	Round.	$520\,x + 2\,580 = p$
10	Blue clay	Square.	$490\,x + 2\,250 = p$

In conclusion, it may be stated that the results used in this discussion represent only one comprehensive investigation. It is felt that many such investigations must be carried out before the theories presented may be referred to as principles or laws of soil action. It could hardly be said that anybody understands thoroughly the principles of soil action. "The bulk of the work, the systematic accumulation of empirical data, remains to be done"; and, after that, some of the theories of soil action may become principles and others will be discarded.

FRANK S. BAILEY,* ASSOC. M. AM. SOC. C. E. (by letter).—It is a very disagreeable experience for an engineer to find after a structure has been erected on a site which he has inspected and given tacit approval, that serious settlement has developed.

The writer remembers one well-educated, intelligent, and able engineer who attained considerable distinction in his special field, who had such an experience with a settling tank designed for the treatment of industrial wastes. A part of the tank settled and cracked and the representative of the company for which it was constructed, made such caustic criticism of the engineer that he, in turn, was filled with resentment and anger. This engineer died at a comparatively early age and a promising career was cut short.

Another well and favorably known engineer who reached still greater distinction, had a similar experience on a larger scale when a considerable por-

* Structural Designer, New England Power Constr. Co., Boston, Mass.

tion of the sewage treatment plant which he had designed also settled enough to require much expenditure for repairs. This engineer died before his prime, and in both these cases the disappointment and chagrin caused by the imperfection of their finished work may have been contributory causes to their unfortunately early deaths.

Both these engineers may have given and probably did give what they believed to be a sufficiently thorough study to the foundation problems involved in the cases cited, but something went wrong, and while it is generally unprofitable to attempt conjecture on what might have been, it is nevertheless natural for one who knew both these men to think that if studies similar to those recorded in the author's paper had been available soon enough, perhaps still greater precautions would have been taken and no troublesome settlement would have occurred.

The writer is impressed by the statement of such a proficient mathematician as the author is known to be, that,

"Foundation problems, throughout, are of such character that a strictly theoretical mathematical treatment will always be impossible. The only way to handle them efficiently consists in finding out, first, what has happened on preceding jobs of a similar character; next, the kind of soil on which the operations were performed; and, finally, why the operations have led to certain results."

The writer knows engineers who have been responsible for the design of many structures which have been erected and have stood without appreciable settlement. Such favorable results may be in part due to good fortune, but another reason seems to be that it is the habit of these engineers to make a most careful study of the characteristics of the earth on which a given structure is to be built.

Although it cannot be expected that all foundations will be so perfectly designed in the future that no settlements of consequence will occur, it is certain that Professor Terzaghi has acquired much new information that can be applied to improvement in foundation practice.

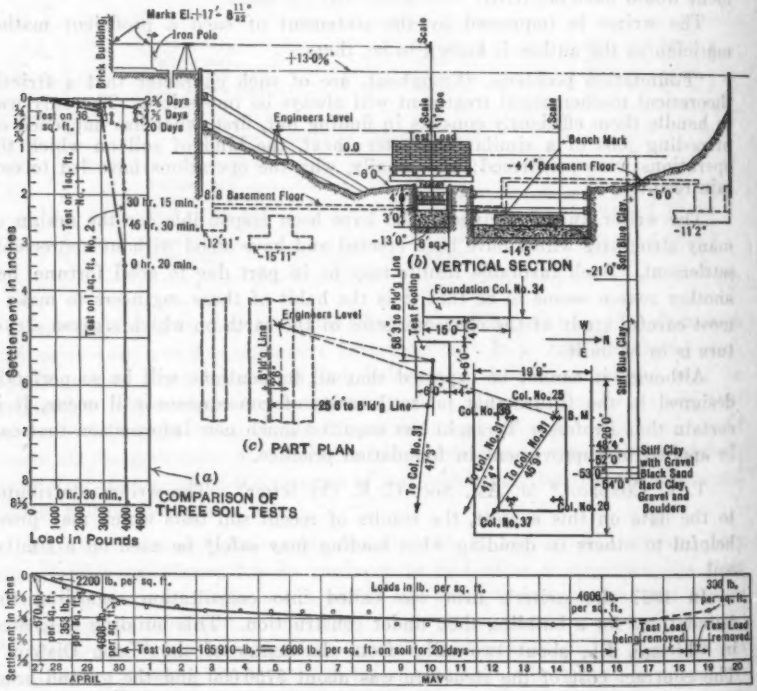
T. L. Condron,* M. Am. Soc. C. E. (by letter).—The writer contributes to the data on this subject, the results of recent soil tests which may prove helpful to others in deciding what loading may safely be used on a similar soil.

In 1927 the writer's firm was called into consultation regarding the foundations for a building then under construction. This building is located in Chicago, Ill., about two miles from the center of the "Loop District". The contract cost of the structure was about $750 000 and the ground area, about 13 000 sq. ft. The building had been designed with "spread footings", resting on clay, from 20 to 30 ft. below street grade, or from 6 to 16 ft. below Chicago City Datum (Lake level). At the time the consultants were called in, the general excavation was nearly completed down to Elevation — 6.0, or 20 ft. below street grade, and five of the footings, or about 20% of the foundations, were in place. Some question had arisen as to the bearing power of the soil and two soil tests had been made by the Contractor

* Cons. Structural Engr. (Condron & Post), Chicago, Ill.

under the direction of the Architect. The results of these two soil tests caused great concern to all. For each test, a load of 4 000 lb. was placed on an area of 1 sq. ft. The settlement under the first test was reported as $8\frac{1}{2}$ in. in 30 min., and for the second test, $2\frac{9}{16}$ in. in 45 hours. The second test was reported as being made in accordance with the recommendations of the Society's Special Committee on the Bearing Value of Soils for Foundations. The results of these two tests as reported to the writer are shown in Fig. 24(*a*), as well as the result of a third test which is described later.

In view of the very unfavorable showing of these two tests, it was proposed to remove the five spread footings that were in place and substitute for all foundations, concrete cylinders extending down to hardpan, at an estimated extra expense of $55 000 to $60 000.



(*d*) DIAGRAM OF SETTLEMENT UNDER TEST LOAD ON 36 SQ. FT.

FIG. 24.

It seemed surprising that the clay, which was fairly stiff and difficult to excavate, offered no better support than was shown by the two tests reported, since they had presumably been made on undisturbed clay at the depth of the proposed footings. However, as the situation was apparently so serious and the expense for the proposed change so large in proportion to the total cost of the project, the writer proceeded to investigate the condition of other

buildings of similar height and character in the immediate vicinity. The proposed building was intended for non-commercial use and was to be only six stories high, except for a tower to enclose tanks. Several buildings were found near-by resting on spread footings designed for a soil pressure of 4 000 lb. per sq. ft. and none of these buildings showed any evidence of unequal settlement although they were from 10 to 15 years old. The footings for the proposed building were to be loaded to not more than 3 000 lb. per sq. ft. Consequently, the consultants recommended that a further soil test be made before arriving at a decision.

In making this test it was felt that, so far as possible, the conditions should be as nearly identical as possible with those for the designed footings. Consequently, a location was selected (see Fig. 24 (*c*)), where a test footing 6 ft. square could be built and left in place without interfering with any part of the general foundation work. An excavation was made from Elevation — 6.0 to Elevation — 13.0, a depth of 7 ft. The upper 4 ft. of this excavation was sheeted and braced and the lower 3 ft. carefully cut to size and filled with 1 : 2.5 : 3.5 concrete (Fig. 24(*b*)). The lower end of a 1½-in. steel pipe, 20 ft. long, to which a gauge was to be attached for readings, was embedded in the concrete block. When the concrete had hardened, a timber grillage was built on which was placed a timber platform to receive the test load of pig iron. Bench-marks were established on one of the footings in place and on near-by buildings and light poles. The arrangement of this test and its location with reference to two footings then in place and to proposed footings, is clearly shown in Fig. 24(*b*) and Fig. 24(*c*). On this diagram is also shown to scale the result of a typical boring which disclosed "blue clay" increasing in stiffness down to Elevation — 52.0, where clay mixed with gravel was found.

Careful level readings were taken on this test footing as the test load was applied, as shown on Fig. 24(*d*), the initial reading being taken when the load of the concrete and platform was 23 970 lb., or 670 lb. per sq. ft. on the soil. On the third day the total load, including the pig iron, was 165 910 lb., or 4 608 lb. per sq. ft. on the soil. At that time the settlement recorded was one-thirty-second more than ¾ in. After the test load had been in place 12 days, the total settlement was measured as one-thirty-second more than ¾ in. and no further settlement could be measured during the next 7 days. The test load was then removed and a rebound of ⅛ in. was recorded. It is interesting to compare the results of this test with the results of the first two tests, as shown in Fig. 24(*a*).

In the writer's opinion this test warrants the conclusion that 3 000 lb. per sq. ft. would be an entirely safe load on this soil. The client was so advised and work was resumed on the building. The full weight of the building has now (1928) been on the foundations for a month or two and no measurable settlement has been reported during the period of construction.

LAZARUS WHITE,* M. AM. SOC. C. E. (by letter).—In the Nineties, when the writer was an undergraduate student, he was thoroughly imbued with the classical methods of computing earth pressures, bearing values of soil,

* Pres., Spencer, White & Prentis, Inc., New York, N. Y.

distribution of pressures, and pile-driving formula commonly taught—Rankine, Baker, Cain, and Wellington—and, after his graduation, he set about to apply them. Fortunately, he was also imbued with the scientific philosophy of Tyndall and Huxley—to accept no theories or formulas unless borne out by experiment. Huxley stated that one clear experiment not accounted for by the accepted theory might forever upset that theory.

The writer's first foundation experience, involving much wood pile-driving, had a disastrous effect on his faith in the *Engineering News* formula. No single experiment conformed to it and to "hit the target" of an observed result, was like shooting at a distant pin with a blunderbuss. Later, during subway construction, he observed that contractors completely and successfully ignored Rankine and Cain in timbering work, and "got away with it" in so many instances that by no scientific philosophy could their theories be justified. Again, his faith in Baker's "Masonry" was upset by the observation that uniform loading did not produce uniform settlement. Hence, with practically all the classical notions upset, the writer decided to observe foundation phenomena for himself and to accumulate data bearing on a new science of foundations.

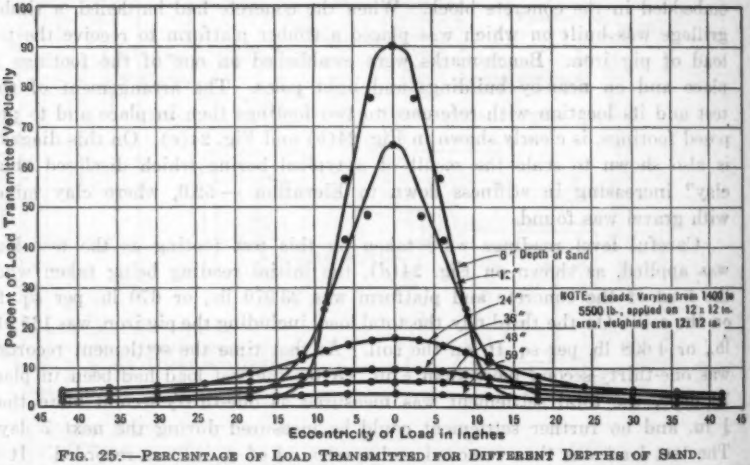


FIG. 25.—PERCENTAGE OF LOAD TRANSMITTED FOR DIFFERENT DEPTHS OF SAND.

In 1914, during extensive underpinning operations along William Street, in New York City, many steel cylinders, 14 in. in diameter, were driven by hand hydraulic jacking into ground composed of varying mixtures of sand and clay. This afforded an opportunity to measure accurately the pressures applied. During the work the elastic properties of earth were observed, especially the rebound after releasing the load, and these were plotted by the engineers of the Public Service Commission of New York.* In 1915, Mr. J. A. Moyer and Professor Fehr, at Pennsylvania State College, published the results of tests† in the distribution of soil pressures measured at various

* *Engineering News*, Vol. 85, No. 27, p. 1268, Fig. 2A.
† *Engineering Record*, Vol. 71, No. 11, March 13, 1915, p. 330.

depths and various eccentricities in relation to the applied loads. (See Fig. 25.) Using these observations as a basis, John F. Greathead, Assoc. M. Am. Soc. C. E., Section Engineer in the days of the William Street Subway, plotted lines of equal vertical pressure beneath a 13½-in., circular plate,* disclosing in a manner which can be readily grasped the general distribution of pressure beneath a footing. This he called the "bulb of pressure"; so far as is known, it was the first time such a diagram was published. (See Fig. 26.) Due to the manner in which the experiments were made—the pressure being recorded on an area equal to the area originating it—Fig. 26 does not show the variations in pressure immediately beneath the footing.

M. L. Enger, M. Am. Soc. C. E., published another bulb of pressure and, because of the use of smaller recording areas, was able to show variations in pressure immediately below the footings. (See Fig. 27.) In general form his diagram is similar to that of Mr. Greathead, but it is more accurate.†

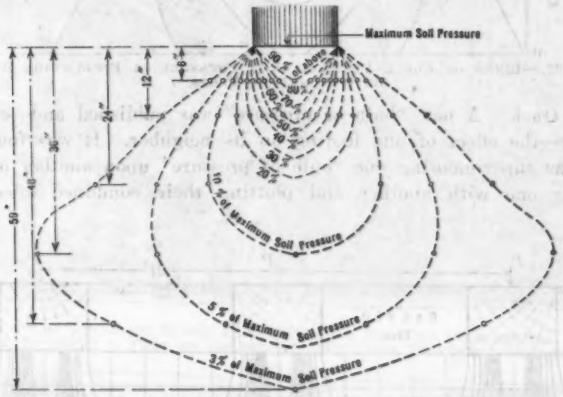


FIG. 26.—LOAD DISTRIBUTION BY BULB OF PRESSURE.

At the same time Professor Enger discovered the great variations in intensities of pressure under footings of different areas, as compared with average unit loads. His formula for such variations is $p = 91 \frac{d^{1.86}}{h^{1.95}}$, in which, p is the ratio (percentage of average unit load) at the depth, h, immediately below the center of the footing, and d is the diameter of the footing. This is of great interest, because if it is assumed that the soil has elastic properties and that a stress-strain relation exists (the applied loads giving the stress and the settlements, the strain or compression), the bearing value of large footings cannot be directly proportional to their areas.

Later, the Joint Committee on Stresses in Railroad Track of the Society and the American Railway Engineering Association in its report,‡ showed the stresses in ballast below railroad ties as ascertained by an elaborate

* *Engineering News-Record*, December 30, 1927, Fig. 3, p. 1037.
† *Engineering Record*, Vol. 73, No. 4, January 22, 1916, p. 106.
‡ *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 1545.

apparatus. Of special interest was a pressure capsule by which intensities of pressure at various points through a mass of ballast could be determined simultaneously. This report is a mine of information for the foundation engineer, especially the full demonstration of the elasticity of the soil beneath

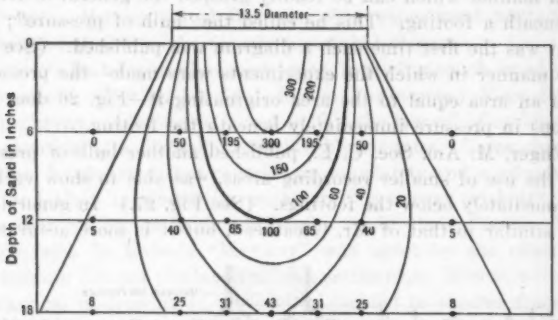


FIG. 27.—LINES OF EQUAL VERTICAL UNIT PRESSURE IN PERCENTAGE BASED ON 4-INCH PLUG TESTS.

a railway track. A new "bulb of pressure" was published and—even more interesting—the effect of one footing on its neighbor. It was found to be the same as superimposing one "bulb of pressure" upon another or, rather, overlapping one with another and plotting their combined effect. (See Fig. 28.)

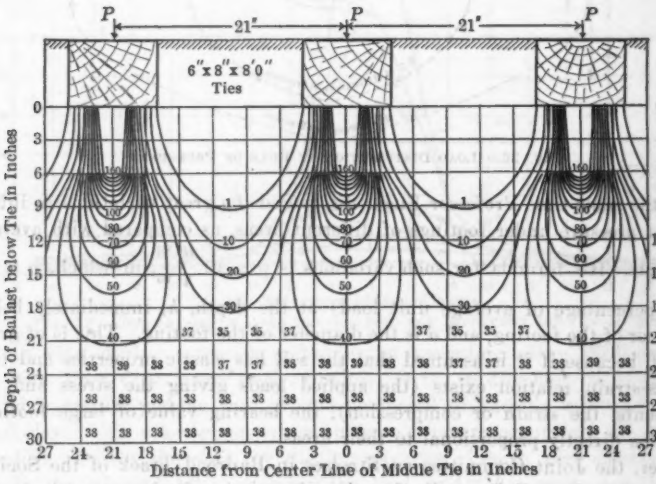


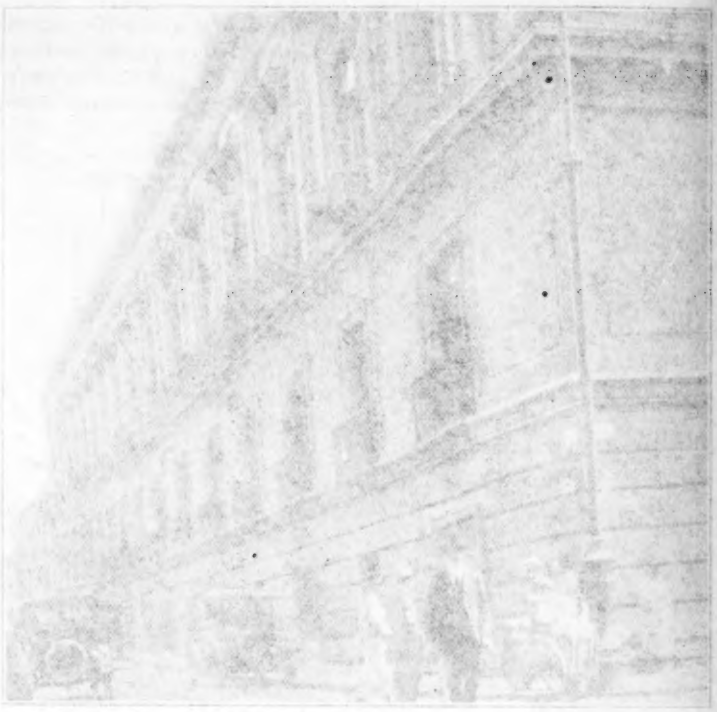
FIG. 28.

In 1925, A. T. Goldbeck, Assoc. M. Am. Soc. C. E., published* a curve indicating that the bearing values of footings or loads for equal settlement

* *Transactions*, Am. Soc. C. E., Vol. 88 (1925), p. 264.

FIG. 29.—VIEW OF SCHOOL OF MINES, CITY OF MEXICO, SHOWING DIFFERENTIAL SETTLEMENT OF ABOUT 3 FEET AT CENTER OF A BUILDING UNIFORMLY LOADED.

FIG. 30.—VIEW OF NATIONAL THEATRE, CITY OF MEXICO, SHOWING 6-FOOT SETTLEMENT OF BUILDING AND ENTIRE AREA AROUND IT.



were not proportional to areas, but to diameters. Mr. Goldbeck invented a pneumatic pressure gauge, with electrical contacts,* which gave very accurate results. For bearing blocks of 0.371 to 8.33 sq. ft., the device checked the Enger formula quite closely.

In the writer's experience, buildings on compressible soil do not settle uniformly, but in parabolic or catenary curves; the center of the building settles much more than the perimeter. This is particularly marked in the City of Mexico where numerous structures show the characteristic curve in their courses. The reason for this should be apparent from data given herein. Since buildings exert pressure on huge areas, the intensity of pressure at the center is very great, and they cause a corresponding compression of the ground, so that the building finally assumes a dished shape. (See Fig. 29.) The fallacy of the old assumption that structures, uniformly loaded, settle uniformly, is thus graphically shown. Fig. 30 shows the settlement of a building with the entire area around it. The original level of the building was the same as the sidewalk. The total settlement was approximately 6 ft.

The Society's Special Committee on Bearing Value of Soils for Foundations, etc., has performed its work painstakingly, but in a field very difficult and rather barren of results; that is, soil classifications and nomenclature to define soils. An infinite number of definitions can be made and yet one may be "left in the dark". Professor Terzaghi has taken foundation engineers "out of the woods" as far as nomenclature is concerned, and, by establishing a few simple physical classifications, it now seems possible to evaluate soils in terms of bearing power.

In describing the characteristics of clay and its behavior in the presence of water due to viscosity and surface tension, he performed a valuable service. Recently, in Detroit, Mich., while excavating in clay and tapping wet areas under pressure, subsidence of surrounding buildings was observed which conformed to the principle laid down by Professor Terzaghi. A wet clay layer, in which water was supplied from seamy bed-rock below, when tapped at a depth of about 100 ft., yielded a considerable quantity of water. Simultaneously with the pumping of this water, settlements of neighboring buildings, several hundred feet away, were noted. Through the underlying rock the water beneath the subsiding area was partly drained—thus causing shrinkage in the wet clay layer above, which was equivalent to superimposing a load sufficient to squeeze the same volume of water out of the clay.

Another effect, noted at Detroit and Albany, N. Y., was that the wrecking or removal of buildings was followed by slight settlement of neighboring structures, where the region was underlaid by a stratum of wet clay.

It has been long observed that buildings on compressible soil do not settle uniformly as to time, but rapidly at first and then more and more slowly. This has been vaguely ascribed to the fact that water is squeezed from the saturated soil below; but by means of the "thermodynamic parallel", Professor Terzaghi has been able to evaluate this phenomenon and to plot a theoretical time-settlement curve of great value. This also has a geological application to alluvial deposits, and indicates that thousands of years may

* *Proceedings*, Am. Soc. for Testing Materials, Vol. XVI, p. 309, 1916.

elapse before an alluvial deposit is consolidated. In this connection the geology of a stratum is of great significance in locating valuable structures. Nearly all old geological strata are fully consolidated and, therefore, have great bearing capacities, whereas recent alluvial deposits bordering streams, old ponds, or lake beds are dangerous. The latter are not fully consolidated, and their excess of water beyond that which is normal for the old strata is not yet squeezed out. Although the deposit may seem stable, an additional load will cause a flow of water and a subsidence or settlement. In some situations a trained geologist may be a better guide than the engineer, even if the latter has had much experience with foundations, and none in theoretical geology.

To illustrate how a structure settles, the information given in Figs. 31 and 32 is very interesting. Fig. 31 is the plan of a building on which settlements were observed between 1920 and 1924. Fig. 32 (*a*) to Fig. 32 (*g*), inclusive, illustrates the catenary curves of walls resting on compressible soil. The various curves show that the time settlements are rapid at first and then become slower.

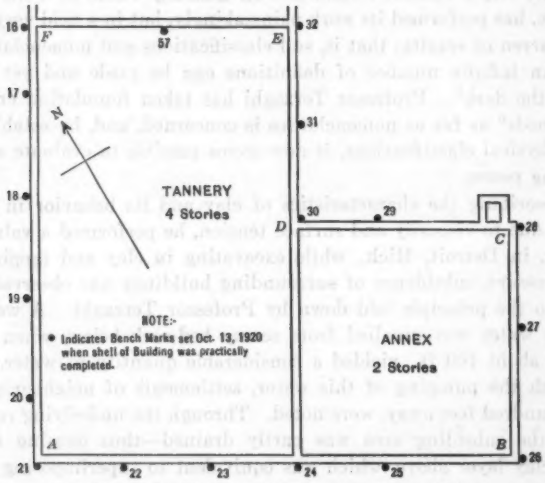


FIG. 31.—PLAN SHOWING LOCATION OF BENCH-MARKS.

The materials beneath the building were thoroughly explored. All test holes (see Fig. 32 (*b*)) were drilled 7 ft. south of the face of the south wall (*A-B*) and about 1 ft. inside the curb line. At Bench-Mark 20 (Fig. 32 (*b*)), a pipe was pushed to about 37 ft. Light driving sent it down to 41.7 ft. At this point the penetration was about $\frac{3}{4}$ in. per blow until, at a depth of 47.2 ft., the last twenty blows sent the pipe down $\frac{5}{16}$ in. and the driving was stopped.

At Bench-Mark 24 (Fig. 32 (*b*)), a pipe was pushed to 33.7 ft. before it was necessary to strike a blow. Then, for each successive increment of twenty blows, the penetration was as follows:

Total penetration, in feet.	Penetration for twenty blows, in inches.
35.5	6.5
40.7	4.5
46.1	3.5
46.9	2
48.0	1
48.29	0.19

After a total penetration of 48.29 ft., the next twenty blows failed to move the pipe. A 12-lb. sledge-hammer was used for driving.

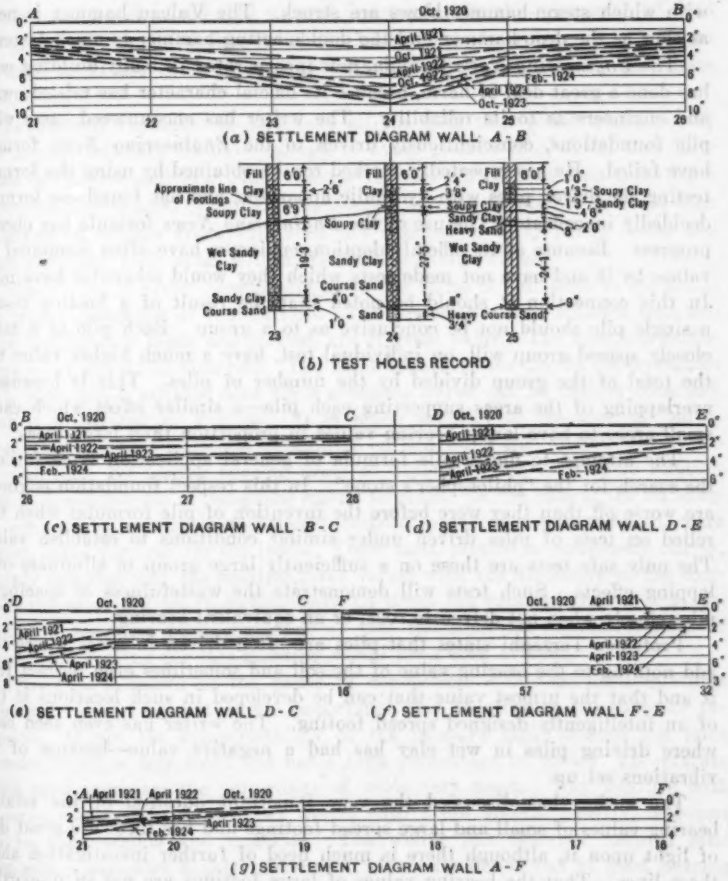


FIG. 32.—RECORD OF SETTLEMENTS OF BUILDING IN SHEBOYGAN, WIS.

The author has dealt the *Engineering News* formula "some staggering blows". Would that it had been "knocked out" completely. It was originated

by Wellington, who really intended it to be applied only to uniform materials like sand, with the drop-hammer then used. He introduced constants to give it a reasonable value. Theoretically, it has a factor of safety of 6, that is, a wooden pile which, by the *Engineering News* formula, gives a working value of 15 tons, has an ultimate value of 90 tons, which is absurd for a small wooden pile.

To cover the use of steam hammers, then coming into use, Wellington arbitrarily changed his denominator, $S + 1$, to $S + 0.1$, giving no good reason except that the more rapid blow of the steam hammer ought to give that much better results. As a matter of fact, there is much variation in the rapidity with which steam-hammer blows are struck. The Vulcan hammer is nearly as slow as the drop-hammer and the double-acting hammer is much faster.

The *Engineering News* formula, now incorporated in many building codes, has done a great deal of harm, because its official character has misled owners and engineers as to its reliability. The writer has encountered cases where pile foundations, conscientiously driven to the *Engineering News* formula, have failed. He has repeatedly checked results obtained by using the formula, testing the driven piles with hydraulic apparatus, and has found the formulas decidedly inaccurate. The use of the *Engineering News* formula has checked progress. Because of its official adoption, engineers have often computed pile values by it and have not made tests which they would otherwise have made. In this connection it should be noted that the result of a loading test on a single pile should not be conclusive as to a group. Each pile of a rather closely spaced group will, on individual test, have a much higher value than the total of the group divided by the number of piles. This is because of overlapping of the areas supporting each pile—a similar effect which causes small areas to have larger bearing values in proportion than large areas.

The attempt to find a pile formula of general application is as futile as the search for the "philosopher's stone". In this respect foundation engineers are worse off than they were before the invention of pile formulas when they relied on tests of piles driven under similar conditions to establish values. The only safe tests are those on a sufficiently large group to eliminate overlapping effects. Such tests will demonstrate the wastefulness of spacing of piles closely when not driven to rock, or an equivalent bearing.

Professor Terzaghi states that piles are often driven in areas where they add nothing to the bearing value of the soil and sometimes even detract from it and that the utmost value that can be developed in such locations is that of an intelligently designed spread footing. The writer has even seen cases where driving piles in wet clay has had a negative value—because of the vibrations set up.

The author has discussed the very important question of the relative bearing values of small and large spread footings and has thrown a great deal of light upon it, although there is much need of further investigation along these lines. That the bearing values of large footings are not in proportion to their areas as compared to small footings is to the writer, sufficiently proved. He has encountered many cases where settlements of large footings were much more than those for smaller ones. In a case where a high building

was founded on a soft coral rock, the smaller footing settled about ½ in. and the larger, 6 in., although in this case the larger footings were designed with lighter unit loads than the smaller ones. Similar results were observed in buildings underlaid by peat, soft silt, etc.

In designing spread footings the old and easy assumption for the designer (that all that is necessary is to assume a working unit load—so many tons to the square foot—and then to divide this into the column loads to get the areas of spread footings) has led, and will continue to lead, to failures. The heavier the loads the more conspicuous the failures. Account must be taken of the shape of the footings, their relative sizes, their spacing, whether or not they are too close together to work independently, the proportion of dead and live load, etc. On this point the writer begs to differ with Professor Terzaghi as to the value of loading tests. If intelligently made, they have great value.

Foundation engineers do not have all the data they could wish—not by any means; but they do have enough to avoid pitfalls into which they have fallen in the past and to produce designs far more intelligent than those heretofore considered good practice. Engineers may even decide not to build high buildings on certain areas with soils not adapted to carry heavy loads, and there are many such areas within the limits of large cities.

The author is rather pessimistic as to the value of soil tests as an aid to the proper designing of foundations. The writer, however, believes that soil tests, intelligently made and plotted, are of great value. The complete settlement curve should be obtained. The ideal apparatus for this is the hydraulic jack, with which reliable data for a complete curve can be rapidly obtained. This is illustrated in Fig. 33. It is much superior to the method* advocated by the Society's Special Committee on Bearing Value of Soils for Foundations, etc. By testing bearing areas of different sizes, a valuable relation may be found for use in designing spread footings. In the case of the building on coral rock, previously mentioned, bearing areas of 1 and 2 sq. ft. were tested, using pig iron for the loads. These tests, plotted as settlement curves, plainly revealed that the bearing value varied with the diameters of the footings, the area of 2 sq. ft. having only 1.4 times the value of that of 1 sq. ft. Had the designers been cognizant of this relation, the impracticability of any spread footing for this building would have been evident.

The results shown in the paper in connection with sand-mica mixtures are most interesting, but undue importance may be given to the shape of the grains. Is not the compressibility of the mixture high because the mica powder itself is very light and has a high percentage of voids? Its mixture with sand increases the percentage of voids very much. Any mixture with a large voids-ratio, or, in other words, with a low specific gravity, is bound to be compressible and unstable. The writer has observed natural sand deposits, apparently highly micaceous, which are dense and of good bearing value, although it may be that actual measurement would show the percentage of mica to be small.

* *Proceedings*, Am. Soc. C. E., March, 1922, Papers and Discussions, p. 527.

The writer does not wish to disparage the use of sand-mica mixtures to simulate earthy materials with various voids-ratios, but wishes to express an opinion that the compressibility of soils is due more to lack of consolidation or to abnormally low specific gravity for the material rather than to the presence of mica or flat grains, although that is a factor in compressibility.

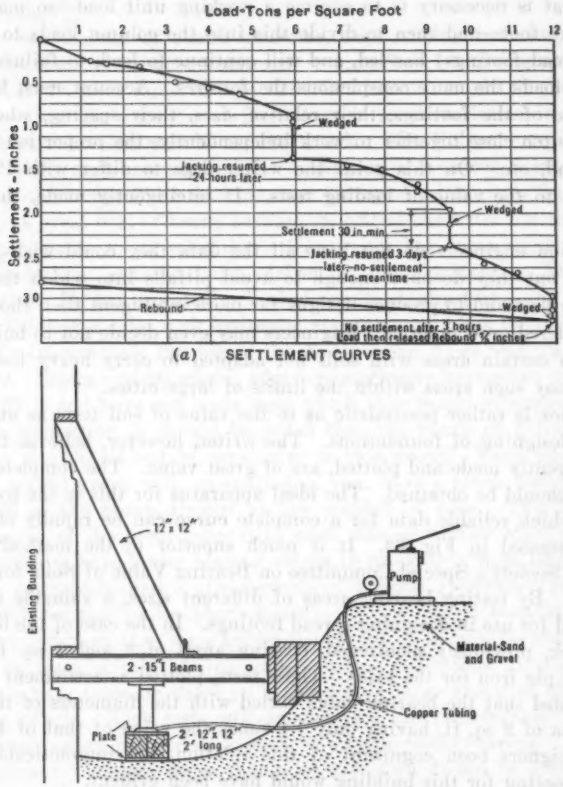


FIG. 33.

In the paper the three properties that determine the behavior of the soil in the foundation pit are given as: First, the volume change produced by an increase of pressure acting on the soil, etc.; second, the permeability of the soil, etc.; and third, the cohesion or shearing resistance of the soil.

The writer would like to add a fourth, which may perhaps be a combination of the first two, but which is nevertheless important and easily ascertained; that is, the density of the soil or rock in relation to an old and well consolidated soil of similar nature. For instance, if the material is sand, determine its weight as compared to a sand of an old and well consolidated geological

period, and if it is clay, find a similar relation. A very wet, compressible clay will necessarily be lighter because of its water content than a dry well consolidated clay of an old period; silt will be very light when compared to a consolidated earth, and, similarly, a coral rock will be very light compared to an old limestone. This relation is not hard to find, because old earths and rocks have a surprisingly uniform specific gravity.

The author draws a very sharp distinction between cohesive and cohesionless soils. That there are great differences in cohesiveness between different soils is not open to question, but in the writer's experience there is quite a residue of cohesion in all soils. Even clean sand such as that suitable for concrete will break in the bank with a curved fracture and stand vertical for a certain height. The writer is inclined to doubt the great effect of depth on settlement, unless the added excavation reveals a decided improvement of the ground. For instance, Professor Terzaghi states that, for a footing in perfectly cohesionless material, a depth of $\frac{t}{d} = 1$ reduced settlements to only one-third of what they would be if the footing rested on the surface of the ground.

Engineers are greatly indebted to the author for his discussion of the distribution of soil reaction over rigid loaded slabs. He has shown clearly (for the first time to the writer's knowledge) the difference between the result as ordinarily computed with the common theory of uniform pressures and that of the much more correct theory of Professor Enger. A great difference is shown, the more correct theory yielding greater bending moments. This may explain why, in Detroit, a concrete mat supporting a large office building on clay is reported to show numerous cracks indicating failure. Single spread footings, such as the ordinary case of an individual footing for each column, the loads being concentrated under the center of application, will yield smaller bending moments than under the theory of uniform loading.

The writer feels with Professor Terzaghi that the future of foundation engineering as an applied science is decidedly encouraging and also that engineers are merely on the threshold of this science. The principle obstacles to progress having been removed, that is, various century-old assumptions that do not conform to Nature, the engineer can now go ahead.

Much can be added to the present knowledge of foundations by applying the methods originated by Professor Enger and Mr. Goldbeck for determining soil pressure. By placing pressure capsules in the soil beneath actual footings the distribution of stress can be determined so that it is possible to compute correctly the bending moments in footings. The relation of depth of footing to settlements and that of areas of footings to settlements, can also be determined, if not by this method then by direct loading tests on footings of different sizes and at different depths. By the use of hydraulic apparatus it is possible to obtain a wide range of tests in a limited time. By this method the effect of various shapes of footings on bearing capacities and settlements may also be found, and by testing groups of piles singly and in combination, the investigator can determine overlapping effects, etc., the proper spacing of

piles for the best results, and the relation between the bearing capacities of a single pile and a group.

In the past engineers were content to state that a building settled very little, but accurate levels recorded, with comparatively little cost, would yield valuable and surprising data. In the future, various representative buildings should be chosen for the purpose of making accurate observations as to settlement. Professor Terzaghi is correct; foundation problems are of such a character that strictly theoretical mathematical treatment will always be impossible, but by systematically accumulating field data it will be possible to approach this ideal much more closely.

The writer hopes that the problem of soil classification will soon be solved along the lines suggested in the paper. He feels that the Engineering Profession is much indebted to Professor Terzaghi, first of all for uprooting certain persistent prejudices and then for leading engineers forward in new directions toward "the promised land" of a real science of foundations.

R. D. N. SIMHAM,* ASSOC. M. AM. SOC. C. E. (by letter).—The writer desires to limit his observations on this interesting paper to the most important question of settlement of soils and its relation to the area of bearing and intensity of loading. It is a fallacy to generalize from experimental results without thoroughly understanding the premises on which, and the objects for which, the experiments are conducted. There are limitations to experimental researches that require corresponding limitations to be made in the laws deduced from them.

In the first place, any results that have been determined by individual experiments under limited considerations and very often in a slipshod manner, that is, without any attempt to follow a general or comprehensive plan and co-ordinated line or method of experimentation, cannot be complete evidence on which to establish any general or fundamental law of science. The writer has learned from experience that any attempts to deduce, from an uncoordinated set of experiments, some sort of a general formula or law that would explain the entire phenomena of soil behavior, has invariably raised serious complications. Therefore, experiments can afford no relief to the present imperfect state of the science of foundations unless they are based on: (1) Constructive lines of thought; (2) a thorough realization of the assumed conditions; (3) a careful diagnosis of every factor that influences the behavior of a soil; and (4) a thorough co-ordination of all the information gained. "Knowledge comes, but wisdom lingers."

There is always a tendency to take some plausible results of experiments and immediately deduce laws of relationship between certain factors involved and then apply them to every single condition that occurs thereafter. Whenever inconsistencies are discovered, all kinds of sophistical arguments are made to explain matters, either by suggesting that the variations are exceptional, that they are unexpected, or are due to defective experiments, or other causes, but all the time insisting that the law, once derived by some mathema-

* Town Planning Asst., Madras, India.

tical manipulations of a set of convenient results, must apply to all sets of conditions.

In an ideal soil, uniform in every respect, it is possible to state that settlement is proportional to intensity of loading in some way and to the area in some other way. With regard to the area, the shape of the bearing surface has an influence on settlement; that is, the settlement of a foundation with the same load and bearing area would be different for dissimilar shapes of the bearing surface. Thus, in a simple way, the relationship between settlement of soil, s; load intensity, w; bearing area, A; and the ratio of mean width to mean length of bearing section (or some other relation that would truly represent the effect of variation in shape and the consequent variation of volume of depth of soil affected), r, may be given in the general form:

$$s = \frac{w^x A^y r^z}{k} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (12)$$

In this equation, k is a constant that expresses the characteristics of the soil.

If this may be taken as truly representing a law of behavior for given soils, then it would be easy to direct some experiments along definite lines and to arrive at the range between which the values of x, y, and z vary. However, in the determination of the value of k, the investigator would probably get into difficulties. The writer thinks that it is impossible to secure identical behavior of soils throughout the processes of varying load, area of bearing, and shape. Any settlement may be the result of several different reactions within the soil. There may be (1) a subsidence due to closer re-arrangement of materials of soil during a particular stage; (2) settlement due to plastic or flattening effect at some other stage, or combined with the former; (3) sinking due to escape or lateral flow of water or escape of other loose materials in the composition of soil; (4) shrinking due to compressibility of soil materials; and (5), under extraordinary circumstances, reduction caused by the crushing of the granules or disintegration of soil materials. Equation (12) may perhaps be modified further to cover these factors also,

$$s = \frac{w^x A^y r^z}{k - (c_1 + c_2 + c_3 + c_4)} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (13)$$

In Equation (13), k may represent a value for soil under standard conditions, or a certain condition of soil behavior when settlement is a minimum. c_1, c_2, c_3, c_4, etc., may represent the reductions necessary in the values of k to allow for subsidence, settlement, shrinking, reduction, etc.

Any soil that is confined before loading to the limit of the natural angle of repose, $x\,O\,A$ (Fig. 34), when loaded, slowly or rapidly adjusts itself in different angles until it attains the limiting angle of compactness of soil materials as indicated by the position, OB, in Fig. 34. Beyond this, a loading will cause the soil to flatten to the limiting position of plasticity, $C'C$. During this period, there may not have been any change in the volume of soil, but merely a bulging outward. Then sinking of soil occurs by the escape of water or loose material within the soil, until the limiting angle of permeability or soil escape, $x\,O\,D$, is reached. Further loading creates pure compression of the soil materials, and the soil shrinks to the

limit of compressibility, $E'E$. Beyond this stage the soil sinks by the crushing of the granular shape. Thus, the actual curves of settlement *versus* load, based on experimental results, would seem to illustrate, not one uniform law throughout, but different laws at different stages. Knowing exactly what is taking place within the soil itself, it should be possible to determine these laws independently through carefully planned experiments, and then, by an easy mathematical computation, a combined or general law of relationship might be deduced.

In another way, assuming that the total settlement of the soil is the combined effect of these factors, it may be found convenient to express the relation for determining the settlement in a somewhat better form, thus:

$$s = \frac{w^{x1} A^{y1} \gamma^{z1}}{k_1} + \frac{w^{x2} A^{y2} \gamma^{z2}}{k_2} + \frac{w^{x3} A^{y3} \gamma^{z3}}{k_3} + \frac{w^{x4} A^{y4} \gamma^{z4}}{k_4} \quad \ldots\ldots(14)$$

in which the quantities, $\frac{w^{x1} A^{y1} \gamma^{z1}}{k_1}$, etc., represent the settlement due to the individual effects of each of the factors mentioned.

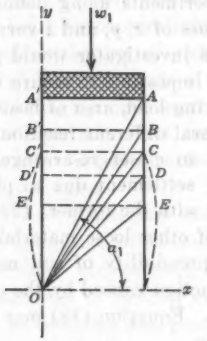


FIG. 34.

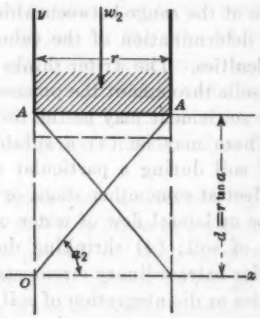


FIG. 35.

There are two more considerations which, instead of increasing the settlement, would reduce it. The side soil protection and frictional resistance offered by the marginal soil should modify the settlement directly below the foundation, and then Equations (13) and (14) will be further generalized to the following forms:

$$s = \frac{w^x A^y \gamma^z}{k - (c_1 + c_2 + c_3 + c_4)} - \frac{V^m}{C} - \frac{P^n}{D} \quad \ldots\ldots(15)$$

and,

$$s = \frac{w^{x1} A^{y1} \gamma^{z1}}{k_1} + \frac{w^{x2} A^{y2} \gamma^{z2}}{k_2} + \frac{w^{x3} A^{y3} \gamma^{z3}}{k_3} + \frac{w^{x4} A^{y4} \gamma^{z4}}{k_4} - \frac{V^m}{C} - \frac{P^n}{D} \quad \ldots(16)$$

In Equations (15) and (16), V and P represent, respectively, the effective volume of soil contributing to the side protection of the compressed soil and the perimeter of the bearing section; while m, n, C, and D are constants.

These do not necessarily comprise all conceivable factors affecting soil behavior. There is no doubt a difference in the behavior of soil under dead

and live load and slowly or rapidly applied load. Eccentricity of loading and character and nature of distribution of thrusts on soil have also to be considered. In fact, it is necessary to know the law connected with each individual factor that influences the soil, and then only can any comprehensive law or laws, which would correctly represent the soil behavior, be attempted. Any such attempt at the present state of knowledge on this subject will naturally be abortive. However, for practical purposes, it may perhaps be sufficient to know what volume of soil will be affected by any load over any bearing area and deduce, with reference to it, some simple relation for the settlement of soil of any specific composition under different loads.

The volume of soil that is directly affected by load is as shown in Fig. 35. This concept does not take into consideration the soil that receives any indirect or virtual effects of load. It limits the volume to that portion lying constrained laterally within the vertical planes bounding the bearing area. The bottom would be defined by a horizontal plane (Ox, Fig. 35) passing through the highest point of intersection of the plane (OA, Fig. 35) of shearing or cohesion and the vertical plane, Oy. The direct settlement of the soil may be considered the result of (1) a closer re-arrangement of soil material; (2) actual compressibility; and (3) reduction due to permeability of this volume of soil or a large proportion of it. Any relation so determined will be useful in a practical way in studying comparative settlements or comparative bearing values of soil.

With these assumptions the relation between settlement and volume for any specific composition of soil may be simply written,

$$s = \frac{w^p\,(c\,V)^q}{k} \qquad (17)$$

in which, V is the volume defined in connection with Fig. 35, and p, c, q, and k are constants. In Fig. 35, let A = the area of the bearing surface; r, the mean or limiting width of soil that restricts the depth, d, of soil affected directly by the load; and a, the angle between the plane of shear or cohesion and a horizontal plane through O. Then,

$$V = A\,r\tan a \qquad (18)$$

Substituting this value of V in Equation (17),

$$s = \frac{w^p\,(c\,A\,r\tan a)^q}{k} \qquad (19)$$

For any specific soil c, tan a, and k will be constant, so that settlement in such case would be represented by the relation,

$$s = \frac{w^p\,A^q\,r^q}{C} \qquad (20)$$

in which, C is a general constant for the characteristics of the specific soil.

As an example, select two kinds of soils to be compared for bearing values. Assume that s, p, c, A, r, and q remain the same in both cases; w_1 and w_2 are the supported loads; a_1 and a_2, the shearing angles; and

k_1 and k_2, constants for the soil in each case. Then,

$$\frac{w_1}{w_2} = \frac{k_1}{k_2}\left(\frac{\tan a_2}{\tan a_1}\right)^{\frac{q}{p}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (21)$$

Equation (21) explains in a simple way the values found by A. T. Goldbeck, Assoc. M. Am. Soc. C. E., with various mixtures of sand and clay.*

FRANK A. MARSTON,† M. AM. SOC. C. E. (by letter).—Considerable progress has been made by Professor Terzaghi in determining adequate and practical methods of classifying soils. For several years, the writer's firm has been carrying on laboratory and field studies on soils with the assistance of and by means of apparatus devised by the author.

In doing such work, there is a real benefit derived from the actual handling of the soil samples. It is not enough to have a trained laboratory man perform certain tests, but, in addition, some work should be done by the engineer responsible for the final decisions in order to acquire the "feel" of the various soils and to observe closely their characteristics as evidenced by varying behavior under test. While the external appearance of a soil sample should be studied, it may be most deceiving as to the actual characteristics and cannot be depended upon solely as a means of grouping soils.

As time goes on, methods and apparatus will be further simplified and their use will be more widespread. The great difficulty at present in the practical use of these improved methods of soil analysis is in the interpretation of the results. Only the gradual accumulation of data will provide the means for accurately translating laboratory results into terms of large-scale field experience. In preparing papers such as this, Professor Terzaghi is rendering the profession a real service by arousing the interest of many investigators. If, as a result of these discussions and investigations, soil studies can be conducted according to standardized methods, records will be made that can be compared and used as a basis for judgment by engineers generally. A tremendous amount of data has been published regarding experiences with foundation conditions, but the lack of a suitable classification for the individual soils has made many of these data of little value.

An illustration can be cited showing the slowness of consolidation of the core material in a certain hydraulic-fill dam, with some data as to the character of the material according to these new lines of classification. All the material for this dam was obtained from a borrow-pit in shale located on a steep hillside which was covered with a layer of disintegrated material 3 to 4 ft. thick. This layer was the principal source of the core material.

Ten samples of the core material were taken at intervals from the top to a depth of 15 ft., at two points (Holes Nos. 1 and 2, Table 7), on the center line of the dam. These holes were 12 ft. and 150 ft., respectively, from the point of overflow of water from the pool during construction.

Samples of core material from the upper parts of the holes were obtained by pushing down a 3-in. pipe and withdrawing it filled with material. This

* *Transactions*, Am. Soc. C. E., Vol. 88 (1925), p. 271.
† Cons. Engr. (Metcalf & Eddy), Boston, Mass.

method was continued until a depth was reached at which the core material was so soft that it would not remain in the pipe. Below this point, the holes were excavated by means of an earth auger which worked with fair success, except that some soft material slid off the auger as it was pulled up. The sampling tubes, 1 in. in diameter and 12 in. long, were used with some difficulty, due to the soft condition of the material which did not always fill the entire length of the tube.

When a ball of the core material was squeezed in the hand, it was found to be so plastic that it flowed out between the fingers. In one test hole, the material at a depth of 12 ft. contained so much water that, after standing over-night, the hole was found to be filled with water to the level of that in the reservoir. Table 7 gives the results of some of the tests of the core material.

There was little difference in the character of the samples from the two holes. The moisture in the samples as taken from Hole No. 1 varied from 19.3 to 42.4% and averaged 33.5% of the weight of dry solids. The samples from Hole No. 2 varied in moisture content from 26.4 to 48.2 and averaged 35.5 per cent. The moisture content did not vary uniformly from the top to the bottom of the holes. The average volume of voids in the samples from Holes Nos. 1 and 2 was 48.6 and 48.0%, respectively, computed on the volume of the wet material as sampled. The specific gravity of the dry core material was 2.748.

A comparison of the liquid limit given in Column (5), Table 7, with the moisture content given in Column (11) (both being expressed in the same terms), shows that about one-half the samples contained more moisture when taken out than at the lower liquid limit.

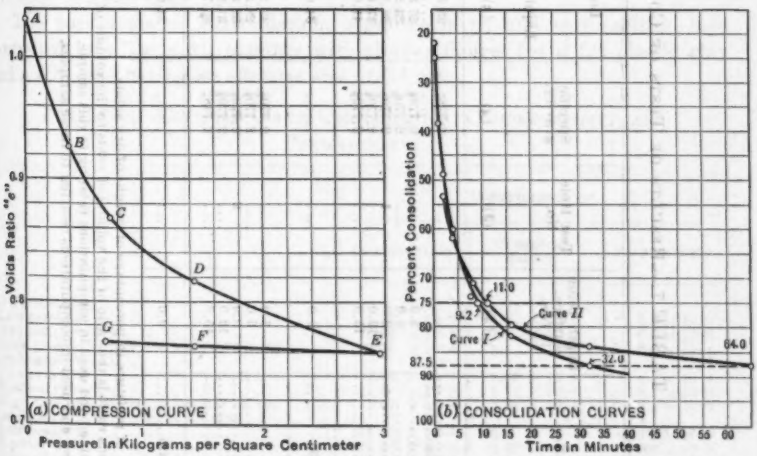


FIG. 36—SAMPLE NO. 712 FROM TEST HOLE NO. 2.

A compression test was made on Sample 712 of this core material for the purpose of determining the compressibility and permeability of the material (see Fig. 36 and Table 7).

TABLE 7.—RESULTS OF TESTS OF CORE MATERIAL FROM AN HYDRAULIC-FILL DAM.

Sample No.	Depth below surface, in feet.	Test Hole No.	Specific gravity.	Lower Limits in Percentage.			Voids Ratio at Limits.†			Moisture content of sample as removed, in percentage of weight of dry solids.
				Liquid.*	Plastic.*	Difference.	Liquid.	Plastic.	Difference.	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
603	....	....	2.778‡	33.5	23.1	10.4	0.931	0.642	0.289	....
(Old dry sample)	....	....	.....	....	....	....	.....	.....	.....	.....
703	3.0	....	2.770§	26.3	23.3	3.0	0.738	0.645	0.093	19.8
704	6.0	....	2.770§	33.2	23.4	9.8	0.917	0.648	0.269	31.3
705	9.0	1	2.770§	32.5	22.0	10.5	0.900	0.609	0.291	34.3
706	12.0	....	2.770§	37.3	28.7	8.6	1.034	0.795	0.239	42.5
707	15.0	....	2.770§	41.0	27.8	13.2	1.135	0.770	0.365	40.2
Average (703-707)	9.0	....	.....	34.06	25.04	9.02	0.945	0.693	0.252	33.5
708	3.0	....	2.770§	42.2	31.5	10.7	1.078	0.872	0.206	48.2
709	6.0	....	2.770§	28.2	20.9	7.3	0.779	0.579	0.200	28.6
710	9.0	2	2.770§	32.3	23.2	9.1	0.895	0.642	0.253	40.5
711	12.0	....	2.770§	31.9	23.5	8.4	0.884	0.651	0.233	26.4
712	15.0	....	2.748‡	35.7	25.2	10.5	0.982	0.698	0.289	34.0
Average (708-712)	9.0	....	.....	34.06	24.86	9.20	0.924	0.687	0.237	35.5

* Weight of moisture in percentage of weight of dry solids.

† The voids ratio is the ratio of the volume of voids to the volume of solids.

‡ Determined and used in computations relating to this sample.

§ Assumed and used in computations relating to these samples.

The dry material was made into a paste and placed in the cylindrical container of the compression apparatus in such a manner as to exclude air and completely fill the space. The paste was approximately at the liquid limit. Pressure was then applied at 0.39 kg. per sq. cm. Observations were made, by means of Ames dials, of the decrease in thickness of the specimen at frequent time intervals until there was no further consolidation. The load was then increased immediately to 0.74 kg. per sq. cm. and similar observations of thickness were made. Similarly, the test was repeated for loads of 1.44 and 2.96 kg. per sq. cm.

When consolidation had been completed, the load was reduced by the same increments by which it had been increased, allowing sufficient time for the sample to expand or rebound to a constant volume between changes in load. The moisture content of the sample was determined before and after testing. Other values used in the analysis were: Specific gravity, 2.748; plastic limit, 25.2 ($e = 0.693$); liquid limit, 35.7 ($e = 0.982$); diameter of the sample, 7.00 cm.; and reduced thickness, 0.705 cm. In Fig. 36 (*b*), Curve I is the consolidation curve when the pressure was increased from 0.74 to 1.44 kg. per sq. cm., and Curve II is the consolidation curve when the pressure was increased from 1.44 to 2.96 kg. per sq. cm.

Point *A* (Fig. 36 (*a*)) represents the voids ratio at the beginning of the test; Points *B*, *C*, *D*, and *E* represent the voids ratios of the completely consolidated specimen under the loads of 0.39, 0.74, 1.44, and 2.96 kg. per sq. cm. Points *F* and *G* represent the voids ratios when the specimen had rebounded after removing loads of 1.52 and 0.70 kg. per sq. cm., respectively. These observations indicate that the material consolidates almost as slowly as a highly colloidal clay.

The coefficients of consolidation, compressibility, and permeability were computed (see Table 8), together with similar figures for a fat plastic clay and a fine sand having an effective size of 0.1 mm.

TABLE 8.—Coefficients of Consolidation, Compressibility, and Permeability.

	Coefficients of.		
	Consolidation.	Compressibility.	Permeability.
Fat Plastic Clay:			
Low pressure	1.72×10^{-3}	4.4×10^{-4}	7.6×10^{-7}
Medium pressure	2.62×10^{-3}	2.3×10^{-4}	6.0×10^{-7}
High pressure up to 2.96 kg. per sq. cm.	2.10×10^{-3}	1.1×10^{-4}	2.3×10^{-7}
Core Sample No. 712:			
Low pressure	2.78×10^{-3}	1.69×10^{-4}	4.7×10^{-7}
Medium pressure	11.7×10^{-3}	0.74×10^{-4}	8.7×10^{-7}
High pressure up to 2.96 kg. per sq. cm.	5.87×10^{-3}	0.38×10^{-4}	2.2×10^{-7}
Fine Sand, Effective Size = 0.1 mm.:			
Low pressure	$171\,000 \times 10^{-3}$	0.5×10^{-4}	$85\,300 \times 10^{-7}$
Medium pressure	$378\,000 \times 10^{-3}$	0.22×10^{-4}	$83\,300 \times 10^{-7}$
High pressure up to 2.96 kg. per sq. cm.	$500\,000 \times 10^{-3}$	0.16×10^{-4}	$80\,000 \times 10^{-7}$

It will be seen that the coefficients of consolidation and permeability for the core material are similar to those of the fat plastic clay and vary greatly

from those of the fine sand. The greater the coefficient of consolidation, the more rapidly consolidation will take place. For the core material, the rate of consolidation is exceedingly slow. The indications are that the core in the dam built with this material to a depth of 15 ft. at least, is in about the same state of consolidation as it was when the dam was built, eight years before the samples were taken.

The influence of grain size on the variation in lower plastic and liquid limits and the difference between these limits is illustrated by data given in Table 9. These data were obtained by separating into its respective portions (according to grain size) a sample from the borrow-pit from which the core material was obtained. This material is a bluish or greenish shale which breaks into bulky pieces.

TABLE 9.—LOWER LIQUID AND PLASTIC LIMITS OF THE SEVERAL FRACTIONS OF FINE MATERIAL FROM BORROW-PIT.

Size, in millimeters.	Proportion of total weight, percentage.	Liquid limit.	Plastic limit.	Difference.
Composite sample	100	33.0	24.7	8.3
0.1 — 0.02	35.2	26.9	29.8	0
0.02 — 0.006	22.2	38.2	36.8	1.4
0.006 — 0.002	17.2	42.2	29.6	12.6
Less than 0.002	25.4	66.8	36.1	30.7

For material between 0.1 and 0.02 mm. in size, the liquid and plastic limits should be practically identical. The discrepancy shown in Table 9 is due to the difficulty in determining accurately the plastic limit in such material. The effective size of the fine material was 0.00075 mm., and the uniformity coefficient was 20.6. The coarsest fraction, 0.1 — 0.02 mm., had relatively low liquid and plastic limits and the difference between them should have been practically zero. The very fine material (less than 0.002 mm.) had high liquid and plastic limits and there was a wide difference between them.

An examination of the borrow-pit material before construction by the methods indicated would have made it possible to forecast the behavior of the material.

ARTHUR M. SHAW,* M. AM. SOC. C. E. (by letter).—This paper is quite unique in that it discusses clay and other soils in the same terms as have been used in the discussion of other materials of construction. While many experimental data have been compiled concerning the bearing power of soils, it ordinarily has not occurred to the engineer dealing with these widely used materials to investigate such properties as tensile strength, elasticity, and especially strength in shear, although the author shows that all these properties are of vital importance in at least some types of soils. The writer was particularly interested in the discussion of the relation of area to the supporting power of soils, having noticed the apparent inconsistent behavior of

* Cons. Engr., New Orleans, La.

weak soils under loads of uniform weight per square foot, but varying materially in area covered.

The following discussion will be limited to muck soils, overlying soft clay, the type of soils commonly found in the "prairie" areas of the lower delta of the Mississippi River.

A number of years ago, it became necessary to place a fuel-oil tank convenient to a pumping plant on a reclamation project near New Orleans, La. The soil, of the type described, had been drained only partly and under a test load (placed on a 3 by 3-ft. platform) appeared to be safe for a load of approximately 500 lb. per sq. ft. A spread foundation was designed to limit the load to 400 lb. per sq. ft., but before the tank was quite three-quarters full, it began to settle unevenly and to an undesirable degree. Hasty shoring brought the tank to a vertical position, an occasional re-adjustment of the shores being necessary to keep it plumb. After about six months of subdrainage, and compacting by the superimposed load, the soil was capable of supporting the full load of 400 lb. without material additional settlement. The total settlement was 2.5 ft.

On a similar reclamation project, experiments were made with the freshly drained muck soil to determine loads which might be used in the construction of light buildings for housing employees. A limit of 250 lb. was adopted and results indicated that this was about correct. General settlement was expected because of the drainage and shrinkage of the muck, but this was fairly uniform and did not cause any serious complications except in connection with one or two buildings which were provided with brick flues. These were carried on short posts (old pile-heads) which rested on a thin local stratum of sand only about 5 ft. below the surface. The considerable separation of grade of the living-room floor and the fireplace has resulted in a novel architectural effect. The Superintendent (who has been in charge since the work was started in 1917) advised that he now (1928) is using a load of 500 lb. per sq. ft. for small footings, but for large footings, he has found it necessary to adopt a considerably lower load limit.

In the construction of levees on similar foundations, the expedient of driving sheet-piles to prevent lateral movement or "bulging" of the muck has been adopted in local practice although it has been found that this is necessary only in extreme cases. Just as good results usually can be secured, and at much less cost, by constructing the levee in multiple layers and by adopting a design that will tend to increase the resistance of the soil to horizontal stresses. Fig. 37 illustrates the section and the method which the writer has used successfully in the construction of levees on exceptionally soft muck lands. The levee section was controlled by the limit of reach of available dredging equipment, as it is seldom practicable to secure a dredge in this section with a reach from the side of the hull in excess of 60 ft. By the use of longer boom dredges, such as were advocated by the writer a number of years ago,* a material increase in height of levees in such soils should be possible. A special type of dredge, which has been in use in California for many years, is well adapted

* "The Selection and Operation of Dredges", *Engineering Record*, December 16, 23, and 30, 1916.

to levee building in soft soils or under other conditions requiring an exceptionally long reach. Following are the principal dimensions of this dredge: Width of hull, 70 ft.; length of hull, 140 ft.; maximum reach from side of hull, 180 ft.; and nominal capacity of bucket, 6 cu. yd.

Repeated experiments have shown that both the spread base and the construction of levees in multiple operations have a beneficial effect in increasing the stability of muck and soft clay soils. With regard to the spread base, the gradually increasing weight, from toe of levee toward the crown, apparently compresses the muck in the zones subjected to only partial loading and renders it more resistant to lateral movement. The gradual application of the load in multiple layers permits the excess water to be expelled from the subsoils and increases their supporting power.

In the lands referred to, there is a great variation in the weight of materials entering into levees, due to different proportions of muck top-soil and clay subsoil. There also is a wide range in weight of the muck soils, some containing a considerably larger proportion of silt than others. As would be expected, the muck soils dry out rapidly after being placed in embankments, above water level, but the "sharkey" clay gives off its water content very slowly. Excavations into clay levees more than two years old have shown a "gummy" condition at points 5 ft. above soil-water level.

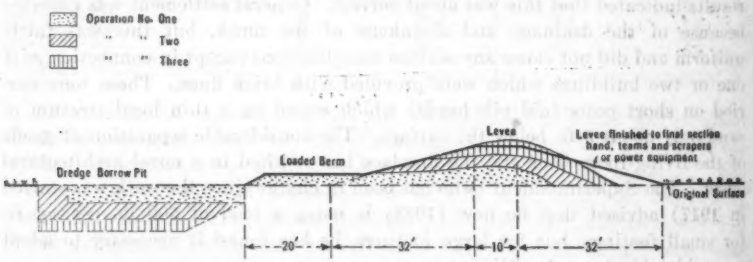


FIG. 37.—CONSTRUCTION OF A LEVEE BY MULTIPLE OPERATIONS.

While concentrated loads, such as those of embankments, may result in failure of the lower strata by a horizontal movement, fills over large areas may be constructed to a considerable height on muck soils with no danger of settlement beyond that caused by the squeezing out of some of the water. The writer has had occasion to make many hydraulic fills over soft muck lands, the added material frequently being clay or sand, but he has never experienced difficulty from excessive subsidence in such cases, provided the fills extended to hard ground or were tapered off gradually at the edges, thus securing the gradual loading mentioned in the foregoing as being desirable for levee construction in similar soils.

Most engineers who have had experience in the construction of railroad embankments across river bottoms have known of localities where embankments 6 to 10 ft. in height could be built without complications, but where a fill of 15 or 20 ft. in height would subside (sometimes suddenly). This subsidence usually would be accompanied by an upheaval of the natural surface,

20 to 50 ft. beyond the toe of the slope. Two such phenomena occurred during the construction of the Illinois Central Railroad from Fort Dodge, Iowa, to Omaha, Nebr., in 1900, one at a point about ten miles north of Council Bluffs, Iowa, and the other in the South Omaha cut-off line, near the bluffs in East Omaha. In the latter case service on the near-by tracks of the Missouri Pacific Railroad was interrupted by the upheaval.

D. P. KRYNINE,* M. AM. SOC. C. E. (by letter).—The author's works† have opened a new era in the study of soils for engineering purposes. Investigations of 10 or 15 years ago generally dealt only with sands, with no consideration for physical properties. Owing to the studies of Professor Terzaghi, the search has penetrated into the interior of clays and sands, the physical properties of which are now being studied by the civil engineer. The present generation is witnessing the birth of a new science, which may be called, "Engineering Soil Science", or "Engineering Soil Theory", and which represents applied soil physics and soil mechanics. It is similar to agricultural soil science; but the latter, by considering principally the influence of water and air on the soil, leaves unstudied the load factor, which is the most important for a civil engineer. Therefore, the establishment of a new science has been inevitable.

Slab Foundations.—In general, the writer agrees with the following statements made by the author:

(1) For soils with great cohesion the settlement produced by a given load increases in direct proportion with the diameter of the loaded area (Fig. 2).

(2) The distribution of the soil reactions over the base of a rigid slab is not uniform. The pressures are equal to zero at the edge of the slab and greatest at the center (Fig. 3).

(3) The settlement of a building is due to consolidation of the soil and to the lateral flow (Fig. 5).

However, there are some objections to each of these conclusions.

First.—Attention is called to a paper by Dr. Ing. Koegler,‡ in which it is stated that the settlement produced by a given load seems to increase directly in proportion with the diameter of the loaded area only to a certain value of this area, after which it becomes more or less constant. In the opinion of Dr. Koegler the elastic condition under a great slab does not permit lateral flow, and the vertical settlement takes place as if the supporting soil were confined laterally. Naturally, the opinion of Dr. Koegler, which seems to be based on his experiments, must be verified.

It also must be noted that his experiments were performed with sands, that is, cohesionless soils. Therefore, the results cannot be generalized until a greater number of experiments with different soils are made; but the example of the settlement of the Standard Oil Building in San Francisco, Calif., leads the writer to believe in these theories. Under a load of 4 800 lb. per sq. ft., the average settlement of the bearing plate was 0.10 in. The settlement of the building should be $152 \times 0.10 = 15.2$ in.; but the actual settlement was only 2 in. If the soil were homogeneous, this fact must be con-

* Prof. of Highway Eng., Moscow Superior Technical School and Moscow Inst. of Transportation Eng., Moscow, Union of the Socialistic Soviet Republics.

† "Erdbaumechanik," Vienna, 1925, and others.

‡ "Die Belastung des Baugrundes," *Der Bauingenieur*, October 27, 1927.

sidered as a proof of Dr. Koegler's statement, but phenomena considered in practice are not as simple and schematic as those studied in a laboratory.

In the example cited the soil was not homogeneous, the settlement of the bearing plate ranging from 0.04 to 0.17 in. The difference is not very great, but in any case there were hard and soft spots in the soil, so that the process of settlement was complicated by the presence of the hard spots which probably produced negative vertical forces acting on the slab. The writer thinks that in the given case and in similar ones it would be better not to deal with the average settlement of the bearing plate, but to elaborate a method of slab design, taking into account the lack of uniformity in the soil.

In regard to the settlement of the caissons at the Chicago Union Terminal, in Chicago, Ill., the writer is not in a position to give a definite opinion on the subject inasmuch as he does not have the description of the work at his disposal. Therefore, he does not know how, where, and when the settlement of the caissons was measured. However, he believes that a caisson cannot follow the law of proportionality of the settlement to the diameter of the loaded area because of the friction against the soil.

Let,

H = the height of the circular caisson, in feet (at the Chicago Union Terminal, $H = 60$);

d = the diameter of the circular caisson, in feet;

f = the average frictional force, in pounds per square foot, of the superficial area of the caisson; and,

q = the load, in pounds per square foot.

Then, the resultant vertical force, q', would be not q, but:

$$q' = q - \frac{f H \pi d}{\frac{\pi d^2}{4}} = q - f \frac{4 H}{d} \quad \ldots\ldots\ldots\ldots\ldots\ldots (22)$$

Equation (22) is an approximate formula which shows that the resultant vertical force acting per unit of the loaded area increases with the increase of the diameter of the caisson. Therefore, the settlement of a large caisson seems to be more considerable than that calculated by Equation (2) not only because of the direct influence of the diameter, but also because of the great resulting force acting on a unit of loaded ground area in comparison with that acting on a smaller one.

Second.—If a slab that is considered uniformly loaded is placed on top of the ground in an excavation, a reaction of the soil develops. Consider only the vertical component of this reaction. The vertical reaction may be confined directly beneath the loaded area (as shown in Fig. 3); or it may be effective beyond this area (see Fig. 38). In the latter case a condition must be satisfied, such that,

$$P + Q_1 + Q_2 = Q_0 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (23)$$

The case shown in Fig. 38 evidently corresponds to the upheaval produced by the vertical component of the reaction. Since it is admitted that the bulging is due to the lateral flow only (Fig. 5), it must be concluded that: $Q_1 = 0$; $Q_2 = 0$. This being the case the derivative of the distribution curve,

$y = f(x)$, at the edge of the slab, which represents the ratio of an infinitely small increase of the reaction force to the corresponding increase of the abscissa beyond the edges, must be equal to zero:

$$f'(b) = 0 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (24)$$

in which, $2b$ is the width of the slab (Fig. 39). As a matter of fact some soil investigations indicate that Equation (24) must be satisfied.*

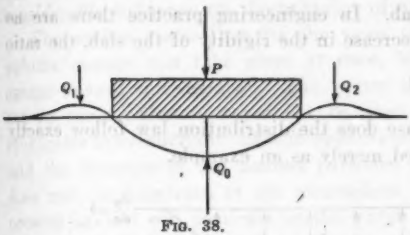


FIG. 38.

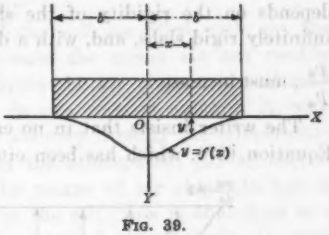


FIG. 39.

As to the curve of pressure distribution, the writer's opinion is that it cannot have a parabolic shape for the following two reasons:

(*a*) The ordinates of a parabola may be calculated accurately according to the equation:

$$y = y_0 \left(1 - \frac{x^2}{b^2}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (25)$$

in which, y_0 is the greatest ordinate in the center; x is the distance from the center of the slab; and $2b$ is the width of the slab. The first derivative of y in Equation (25) is:

$$\frac{dy}{dx} = -y_0 \frac{2x}{b^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (26)$$

which is not equal to zero, when $x = b$.

In order to satisfy the conditions in Equation (24), the shape of the distribution curve may be modified slightly, for instance, so that:

$$y = y_0 \left(1 - \frac{x^2}{b^2}\right)^m \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (27)$$

For any value of $m > 1$, and for $x = b$, the derivative of this function is equal to zero:

$$\frac{dy}{dx} = -\frac{2mxy_0}{b^2}\left(1 - \frac{x^2}{b^2}\right)^{m-1} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (28)$$

Fig. 40 represents two curves placed side by side for comparison. Equation (25) expresses the left half and Equation (27) the right half. Equation (27) recalls that of the error function, or, better, of the second curve of Pearson.†

(*b*) The area of the curve of pressure distribution, as drawn on the author's Fig. 3, represents the load on a strip of slab 1 ft. wide. If the average pressure on the slab is p_a, it may be written:

$$2b\,p_a = \frac{2}{3} \times 2b\,p_0 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (29)$$

* Koegler, "Uber die Verteilung des Bodendruckes unter Gründungskörper," *Der Bauingenieur*, February 5, 1926.

† *Philosophical Transactions*, Royal Soc. of London, Series A (1895), Vol. 186, Pt. I, pp. 360 and 372; also, Fig. V, p. 365.

and, furthermore,

$$p_0 = \frac{3}{2} p_a \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (30)$$

Equation (30) signifies that the pressure at the center of the slab is always 50% greater than the average. In the writer's opinion the ratio, $\frac{p_0}{p_a}$, depends on the rigidity of the slab. In engineering practice there are no infinitely rigid slabs, and, with a decrease in the rigidity of the slab, the ratio $\frac{p_0}{p_a}$, must increase.

The writer insists that in no case does the distribution law follow exactly Equation (27), which has been cited merely as an example.

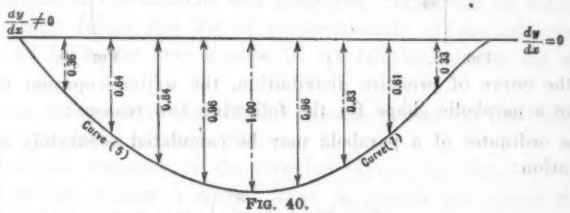


FIG. 40.

Third.—The writer believes that the two parallel lines corresponding to two principal sources of settlement (see Fig. 5) are not really parallel, and that the settlement at the center of a slab is greater than at its edges. For this belief he presents the following reasons: (1) In the middle of a building, because of symmetry, there can be no lateral flow; hence, the deflection in the middle of a slab depends only on the vertical component of the pressure; (2) the vertical pressure at the edge being equal to zero, there can be no settlement due to the vertical pressure; hence, the deflection at the edge of a slab depends only on the lateral flow; (3) if the curve of deflection due to vertical pressure (compression) be drawn, it will represent a curve with ordinates decreasing from the center to zero at the edges; on the contrary the ordinates of the curve of deflection due to lateral flow must increase from zero at the center to a certain value at the edges; and (4) if Koegler's theories are correct, the lateral flow does not increase from the center toward the edges of the slab during the entire time, but stops when $x < b$ (see Fig. 39). Hence, the settlement at the center of the slab seems to be greater than at the edges. The settlement of small rigid disks, 3 or 5 ft. in diameter, naturally cannot give any idea of the difference of settlement in the center and on the perimeter of the slab.

Generally, a foundation slab is a statically indeterminate structure. The distribution of soil reaction depends on the rigidity of the slab. Therefore, bending moments and shearing forces vary with the change in its rigidity. The distribution of soil reactions is intimately bound to the deflection of the slab, although Hooke's law is not applicable to soils. Hence, the soil deflections, reaction distribution, and stresses in the slab are interrelated. This idea is not new. In 1914, Brugsch and Briske published a paper on the

influence of the deflection of the foundation on the design of statically indeterminate structures.*

Entrapped or "Pinched" Air.—Professor Terzaghi greatly merits the approval of the Engineering Profession for demonstrating that clay and water form an intimately connected system. Hence, the mechanics of clay, without considering the influence of water, practically became absurd. The writer desires to show that in some cases not only the water, but also the air, has a considerable influence on the behavior of clay.

The author states that "if the voids of the soil are filled with air, the volume change can take place at once, because the excess air can readily escape toward the surface". The writer thinks the air present in soil may act in three different ways: (1) So that, in one way or another, it communicates to the surrounding atmosphere and assumes directly the temperature and the pressure of the nearest particles of the atmosphere; (2) so that it does not communicate to the atmosphere by means of air channels, but is pressed against soil particles by the water in the soil; this is analogous to a bubble in a spirit-level, and may be termed entrapped or "pinched" air; such air bubbles may often be microscopic in size; and (3) as adsorbed air which sticks to the particles of soil.

The first of these conditions is the one recognized by Professor Terzaghi. As to the second, which naturally must have great influence on the elastic properties of soils, the writer is not sure of its presence in the deep layers of the ancient natural clays. As to relatively modern deposits, however, and especially the artificial fills, there is no doubt as to the presence of entrapped air. Therefore, an investigation of the effect of air and, if necessary, the introduction of the element, "air", into the system, "soil ÷ water", seems to be of definite importance.†

The writer believes that a study of capillarity as applied to water in the soil, leads to the conclusion that there is also a certain quantity of entrapped air.‡ If the clay pores are not completely filled with water, there must be air present. Furthermore, because clay pores represent excessively thin clefts, it is very difficult to imagine that the air may easily escape from them. At any rate, the influence of the entrapped air on the elastic properties of soils should be investigated.

Pile Foundations.—It is interesting to consider the author's views on pile foundations in reference to the pile-driving formula of Professor Guersevanoff, which is widely used in Russia.§ Professor Terzaghi's notations are used with the following additions:

r = rebound of the hammer after the blow; and

a = coefficient of the lost work, depending on the nature of the pile, method of its driving, etc.

* Brugsch and Briske, "Einfluss der Nachgiebigkeit des Baugrundes auf die Berechnung äusserlich statisch unbestimmter Bauwerke," *Beton und Eisen*, 1914, pp. 15, 53, 85, and 183.

† D. P. Krynine, "On the Technical Rôle of Air in Soils" (in Russian), *Transactions*, Inst. of Structural Research, 1928.

‡ D. P. Krynine, M. V. Ivanova, and T. A. Ovsiannikoff, "On the Capillary Rise" (in Russian), *Transactions*, Inst. of Structural Research, 1928.

§ *The Cement* (Russian Magazine), 1916; First issue, p. 2; Second issue, p. 73. This magazine is no longer published.

The work done by the drop of a hammer consists of three parts: (1) Pile penetration; (2) elastic deformation; and (3) lost work; for instance, increase in temperature, etc. This may be expressed:

$$R\,h = [A] + [B] + [C] = Q_d\,s + R\,r + a\,R\,h \ldots\ldots (31)$$

In his analysis, Guersevanoff ignores the term, $R\,r$, so that Equation (31) becomes:

$$(1 - a)\,R\,h = Q_d\,s \ldots\ldots (32)$$

The coefficient, $1 - a = b$, depends on the value of Q_d and the cross-section of the pile, F; that is,

$$b = f\left(\frac{Q_d}{F}\right) \ldots\ldots (33)$$

By making various assumptions he comes to the conclusion that the shape of the curve of Equation (31) is a hyperbolic one:

$$b = \frac{R + m^2\,G}{R + G} \times \frac{1}{1 + \dfrac{Q_d}{F}\,n} \ldots\ldots (34)$$

The coefficient, n, depends both on the nature of the pile and on the method of its driving. For instance, for wooden piles without followers, $n = 10\,\dfrac{K\,g}{c\,m^2}$; for concrete piles with followers, $n = 0.5\,\dfrac{K\,g}{c\,m^2}$. Definitely, the formula, in kilograms and centimeters, is:

$$Q_d = -\frac{n}{2} + \sqrt{\frac{n^2}{4}F + n\,\frac{F}{s}\,R\,h\,\frac{R + 0.2\,G}{R + G}} \ldots\ldots (35)$$

This formula has somewhat the appearance of Equation (3), but Equation (35) neglects partly the length of the pile, and the modulus of elasticity of the pile material enters implicitly into the value of n.

The author's concept of hydrodynamic stresses induced in soils is truly brilliant, but there is some question as to the squeezing of water out of soil beneath the point of the pile. Suppose the pile is driven into clay containing 50% of voids which are completely filled with water. Then, the thickness of the film of water around the pile must be, approximately (deformations of the pile disregarded):

$$T = \frac{0.5\,\dfrac{\pi\,d^2}{4}\,h}{\pi\,d\,h} = \frac{d}{8}$$

in which, d is the diameter of the pile. In Fig. 41 the full line represents the position of the pile before the blow, and the dotted line the position after the blow. Thus, the film of water around a 10-in. pile would be 1¼ in. thick, which seems to be too much. Therefore, the writer thinks that the vertical force transmitted by the pile: (*a*) Compresses the soil, thereby causing the pile to move downward and producing a certain lateral flow of hard particles; and (*b*) acts on the soil water just as in any case of clay loading.* In Fig. 42 the part of the vertical force acting on the water is divided into two components:

* Terzaghi, "Erdbaumechanik," Abschnitt 12, 20, and others.

(1) The horizontal force, moving water away from the pile and thus compressing the entrapped air; and (2) the force parallel to the surface of the pile. The reaction against this component of force is what causes the film to form on the surface of the pile. The posterior absorption of the film by the soil may occur, in the writer's opinion, only if there is air in the voids. The layers of clay in Fig. 42 are assumed to be approximately horizontal.*

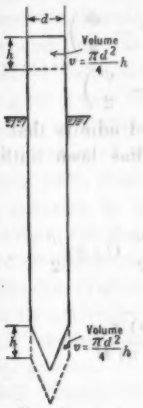


FIG. 41.

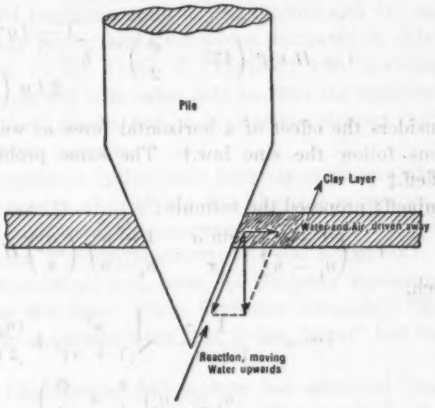


FIG. 42.

Russian Contribution to the Science of Foundations.—The author deals with the present status of the science of foundations as well as with its future.

Russian engineers have made some noteworthy contributions to the theory of pile-driving.† They have also published some works dealing with the determination of the depth, t, of the foundation. Through a paper by Professor Kjürdümoff‡ three formulas, those of Paucker and Jankowsky, came from Russia into European literature.§

Let,

t = the depth of the foundation;

H = the height of a sand prism, with base area and weight equal to those of the building; and,

ϕ = the angle of the internal friction.

Then, Paucker's formula is:

$$t = H\, t g^4 \left(45° - \frac{\phi}{2}\right) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (36)$$

The first formula of Jankowsky is:

$$t = \frac{H}{2}\, t g^4 \left(45° - \frac{\phi}{2}\right) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (37)$$

* D. P. Krynine, "Elementary Proof of Shale-Likeness of Clay Particles," *Public Roads*, January, 1928.

† Jankowsky, "On the Resistance of Pile Foundations," *Journal*, Ministry of Ways of Communication, 1887; Guersevanoff, the previously cited paper, 1916; Dmohovsky, Papers on the influence (*a*) of the geometric shape of piles; and (*b*) of the eccentricities of loads, Works of the Moscow Inst. of Transportation Eng., 1927. There are also other papers by the same author and his textbook on "Engineering Foundations," 1928.

‡ Kjürdümoff, "Zur Frage des Widerstandes der Gründungen auf natürlichen Böden," *Der Civilingenieur* 1892.

§ The derivation of these formulas is to be found in Terzaghi's "Erdbaumechanik," 1925, p. 229.

and his second formula is:

$$H = 2\,t\,\frac{t\,g^2\left(\frac{45^\circ + \phi}{2}\right)}{t\,g^2\left(\frac{45^\circ - \phi}{2}\right)} \qquad (38)$$

Belzetsky's formula* is:

$$t = H\,t\,g^4\left(45^\circ - \frac{\phi}{2}\right) - b\,\frac{1 - t\,g^4\left(45^\circ - \frac{\phi}{2}\right)}{2\,t\,g\left(45^\circ - \frac{\phi}{2}\right)} \qquad (39)$$

He considers the effect of a horizontal force as well and admits that the soil reactions follow the sine law.† The same problem has been outlined by Prokofieff.‡

Miniaeff§ proposed the formula:

$$t = \left(\frac{1+n}{n_1 - n}\right)\left(\frac{\sin\alpha}{\pi} - \frac{1-n}{n_1 - n}\right)\left(\frac{\alpha}{\pi}\right)H - \frac{b}{2}\,C\,t\,g\,\frac{\alpha}{2} \qquad (40)$$

in which,

$$\cos\alpha = \frac{1}{1+n} - \sqrt{\frac{n^2}{(1+n)^2} + \frac{(n_1 - n)\,\pi\,\alpha}{2\,(1+n)\,H}}$$

$$n_1 = t\,g^2\left(\frac{\pi}{4} + \frac{\phi}{2}\right)$$

and,

$$n = t\,g^2\left(\frac{\pi}{4} - \frac{\phi}{2}\right)$$

Equation (40) is based on the following hypotheses: (1) The distribution of stresses in soils follows the same laws as in a solid; (2) "sets" in a dry substance are not functions of forces; and (3) stresses in dry substance are determined only by elastic deformation. The problem consists in determining if, under a given set of forces, an elastic deformation of the soil takes place. If not, a supplementary set of forces must be applied, namely, the weight of the soil above the level of the bottom of the foundation.

Pouzyrewsky‖ considered a condition of the soil when all the deformations are elastic only, and proposed the formula:

$$t = H\,\frac{C\,t\,g\,\phi + \phi - \frac{\pi}{2}}{C\,t\,g\,\phi + \phi + \frac{\pi}{2}} \qquad (41)$$

In his paper on the theory of dry substances, Nemiloff¶ developed a formula which, like the formula of Belzetsky, takes into account the width, b, of the foundation. It is worthy of note that, in 1913, he proposed an apparatus

* Belzetsky, "Statics of Constructions," 1914, p. 95.

† *Loc. cit.*, p. 83.

‡ Prokofieff, "Theory of Constructions," 1928, Vol. II, p. 258.

§ Miniaeff "On the Distribution of Stresses in Dry Substances," Tomsk, Siberia, 1915, p. 85.

‖ Pouzyrewsky, "Foundations Design" (in Russian), 1923, p. 38.

¶ Nemiloff, "Contribution to the Theory of Dry Substances," *Journal*, Ministry of Ways of Communications, 1913, Issue 9, pp. 126–148.

for determining the coefficient of internal friction* based on practically the same idea as that for the apparatus used in 1917 by the Society's Special Committee to Codify Present Practice of the Bearing Power of Soils for Foundations, etc.†

This fact shows that there is a need for a scientific center to look after the technical literature in civil engineering the world over, in order to avoid the duplication of work and to utilize the technical ideas of different nations.

Generally, Russian civil engineers seem to like the soil and dry substances theory, and there are many papers and discussions dedicated to this subject. The writer calls attention to the works of Prilejaeff‡ and Spalving.§ The latter gives a new theory; he not only takes into account the equilibrium of a soil prism as Paucker and others do, but he studies the stresses that develop in the interior of this prism as well.

Since 1922, Russian engineers (principally highway engineers) have given their attention to the necessity of studying the physical properties of soils. The writer was the first in Russia who expressed in printed form the necessity of co-operation between highway engineers and soil scientists.‖ In 1923, the Russian Highway Department and, later, its Highway Research Bureau, began its activities along this line. When Professor Terzaghi's "Erdbaumechanik" appeared in 1925, it became clear that a new "slant" had been given to this science.

Soil Classification.—Engineering soil science has advanced considerably in recent years; but a definable system of soil classification had not appeared until this paper was presented. The research engineer is in a difficult position when receiving a sample of soil from a job. He may tell the builder as much as he wishes about the internal friction of the soil, about its compressibility, its permeability, etc.; but the only question asked is, "How many pounds pressure will this soil support?"

A laboratory cannot produce the answer to this question categorically. The most practical solution of the problem seems to be to elaborate a combination of simple laboratory experiments with a simple routine test in the field.

S. P. WING,¶ ASSOC. M. AM. SOC. C. E. (by letter).—The writer is connected (1928), with the design of a dam that is to be located at what is known as Site No. 7 at Bridge River in British Columbia. This work is at the lower end of a 30-mile glaciated valley that is about ½ mile in width, with surrounding hills of granites and schists. The actual dam site is about 800 ft. in width between granite outcrops. Borings showed that the rock on either side pitched

* Nemiloff, "Contribution to the Theory of Dry Substances," *Journal*, Ministry of Ways of Communications, 1913, Issue 9, p. 140, Fig. 3.

† *Proceedings*, Am. Soc. C. E., August, 1917, Papers and Discussions, p. 1179; and, further, Fig. 1, p. 1174.

‡ Prilejaeff, "Contribution to the Theory of Pressure on Sustaining Walls" (in Russian), 1913.

§ Spalving, "A New Theory of Dry Substances," *Water Transport* (Russian magazine), February, 1927.

‖ D. P. Krynine, "American Methods of Earth-Road Building and Their Application in Russia," *The Messenger of Technics and Economics* (Russian magazine no longer published), January, 1922.

¶ Engr., Constr. Dept., British Columbia Elec. Ry., Vancouver, B. C., Canada.

sharply downward, and drilling in the center was stopped when bed-rock was not reached at a depth of 225 ft.

As borings showed a heavy bed of clay lying about 30 ft. below the surface, numerous other test holes were sunk, and a clear conception of the underlying beds was obtained. At the neck of the valley are beds of coarse talus and gravel, perhaps 200 ft. deep as a maximum, resting on bed-rock. Overlying these strata is a continuous bed of glacial or post-glacial clay perhaps 10 ft. thick at the valley neck, but rapidly deepening to a thickness of 90 ft., or more, up stream and forming an impervious blanket in contact with rock on either side. Overlying this are recent deposits of silt and sand and gravel from 20 to 30 ft. thick, forming the present valley floor.

Tests were made with a view to determining the bearing power and settlement of both the clay and the overlying beds. The amount of settlement is considered of great importance because the project calls for a rock-fill dam with concrete facing and a water-tight apron extending up stream to a point where cut-off can be made into deep clay. The design of this lining will have to provide for any differential settlement.

The soil-pressure testing apparatus was similar to that recommended by the Society's Special Committee on the Bearing Value of Soils for Foundations, etc.,* but was designed for a maximum load of 60 tons, so that loading areas of 1, 2.5, and 4 sq. ft. could be used. In general, the apparatus proved satisfactory, but in future tests an oil-jack would probably be substituted for the screw-jack which was used as a fulcrum, because the force required to move the latter in leveling induced extra settlement of the test post.

Test Pit No. 3.—The first tests were made at Test Pit No. 3. Fig. 43 shows the log of the test hole at this location. The material was sand and was composed of sharp angular fragments, largely quartz. Eight feet of the top-soil was removed and the tests on different areas were made in the bottom of the open pit on the undisturbed bed of sand. The mechanical analysis of this sand was as follows:

Sieve No.	Percentage retained.
4	1.4
8	4.2
14	6.5
28	15.8
48	42.5
100	26.9
200	2.2
200	0.5†
Total sample	100
Percentage of mica	0.6

This sand was saturated and the pit required sheet-piling and continuous pumping to keep it unwatered. However, during tests the pit was allowed

* *Proceedings*, Am. Soc. C. E., March, 1922, Papers and Discussions, p. 529, Fig. 1.
† Percentage passing.

to fill with water. Figs. 43 and 44 give the results of the loading tests. An attempt was made to measure the flow of the surrounding soil during the loading of the 4-ft. square plate. Stakes were driven 18 in. into the ground at a distance of 1 ft. from the loaded area. These did not show consistent results, but at the end of the test they had risen an average of 0.015 ft. This settlement may have been due to the effect of unwatering.

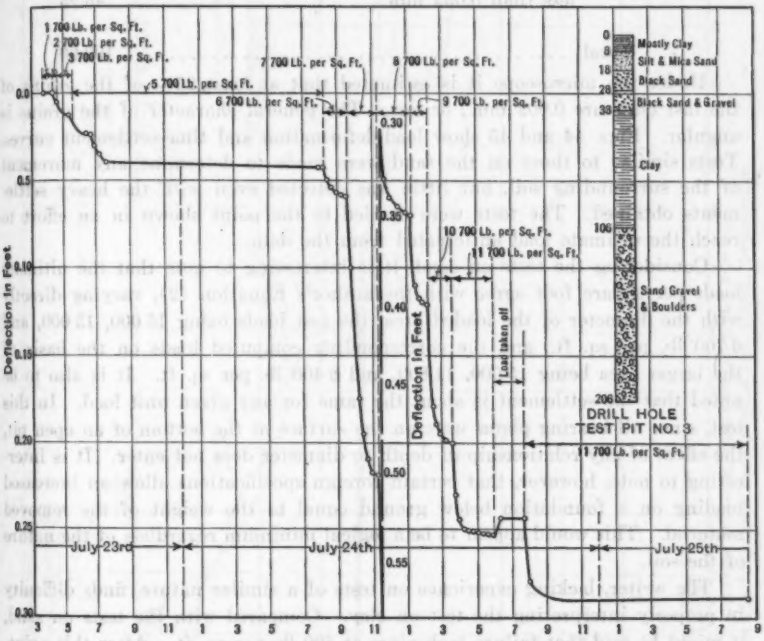


FIG. 43.—TIME SETTLEMENT CURVE FOR SILT AND GRAVEL.

Test Pit No. 2.—Fig. 45 shows the test-hole log at Test Pit No. 2. A pit, 5 by 7 by 13 ft. deep, was sunk, sheet-piled, and pumped dry to a depth 3 ft. below the area being tested and the test was made on a 2-ft. square area of undisturbed clay in place.

The material was clay and was stiff and difficult to spade, but under the men's feet it became plastic. Its characteristics follow:

Weight of clay as sampled, in pounds per cubic foot	112
Percentage of water (in terms of weight of dry material in 1 cu. ft.)	40
Percentage of lower limit of liquid state	42.5
Percentage of lower limit of plastic state	36.6

A change in moisture content from 37 to 26% causes shrinkage of 12% in original volume.

The mechanical analysis by elutriation is as follows:

	Percentage.
Particles, 0.075 to 0.050 mm	0.063
" 0.050 to 0.025 mm	1.32
" 0.025 to 0.010 mm	16.43
" 0.010 to 0.005 mm	35.52
" less than 0.055 mm	45.25
Total	100.00

Under the microscope it is estimated that at least 25% of the grains of the last class are 0.002 mm., or less. The general character of the grains is angular. Figs. 44 and 45 show load-deformation and time-settlement curves. Tests similar to those on the sand were made to determine any movement of the surrounding soil, but little was detected even with the heavy settlements obtained. The tests were carried to the point shown in an effort to reach the ultimate load anticipated from the dam.

Considering the tests on sand, it is interesting to note that the ultimate loads per square foot agree with the author's Equation (2), varying directly with the diameter of the loaded area, the test loads being 15 000, 13 000, and 6 700 lb. per sq. ft., and the corresponding computed loads on the basis of the larger area being 15 000, 11 800, and 8 400 lb. per sq. ft. It is also to be noted that the settlement is about the same for any given unit load. In this test, since all bearing plates were on the surface at the bottom of an open pit, the effect of any relationship of depth to diameter does not enter. It is interesting to note, however, that certain foreign specifications allow an increased loading on a foundation below ground equal to the weight of the removed material. This would appear to be a logical minimum regardless of the nature of the soil.

The writer, lacking experience on tests of a similar nature, finds difficulty in properly interpreting the test on clay. Compared with the tests on sand, it might be said that failure took place at 500 lb. per sq. ft. After this point, any increased load caused heavy settlements but, as the curves show, those decreased with time, and practical equilibrium was established, even with as heavy loads as 12 000 lb. per sq. ft. and total settlements of 2.3 ft., the settlement in the last ten days at this load being only 0.01 ft. It is possible that with this heavy settlement some extra bearing surface of the apparatus became effective, but it is not thought that this was important. Release of the load and its re-application caused some slight additional settlement, as shown by the curve (Fig. 45). In the main it is thought that the increased supporting power of the clay was due to local consolidation under the bearing plate. That similar results will occur with a diameter of loaded area 100 times greater, seems doubtful. As the author's formulas are only applicable to loads within what might be called the elastic limit, they give no guide as to the future behavior of this material. Fortunately, in the present case, a diversion dam of small proportions is to be built first, and it is intended to make more loading tests on the exposed foundation and then to compare those

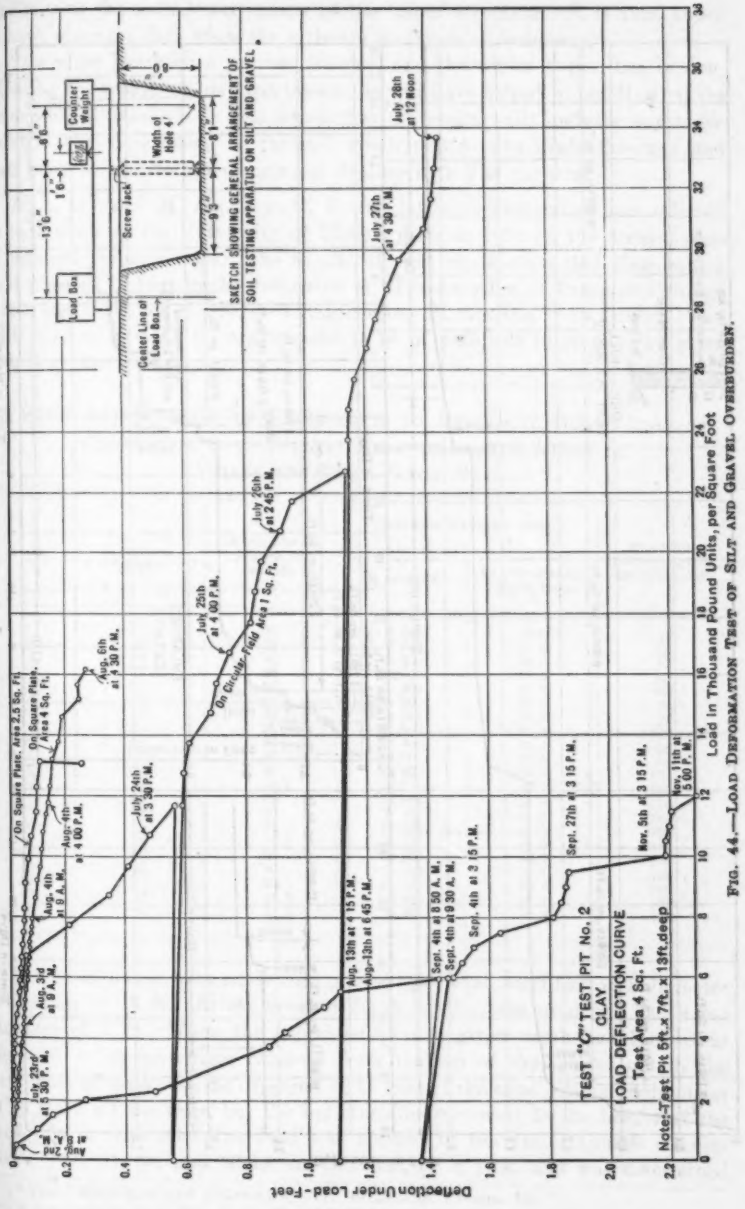


FIG. 44.—LOAD DEFORMATION TEST OF SILT AND GRAVEL OVERBURDEN.

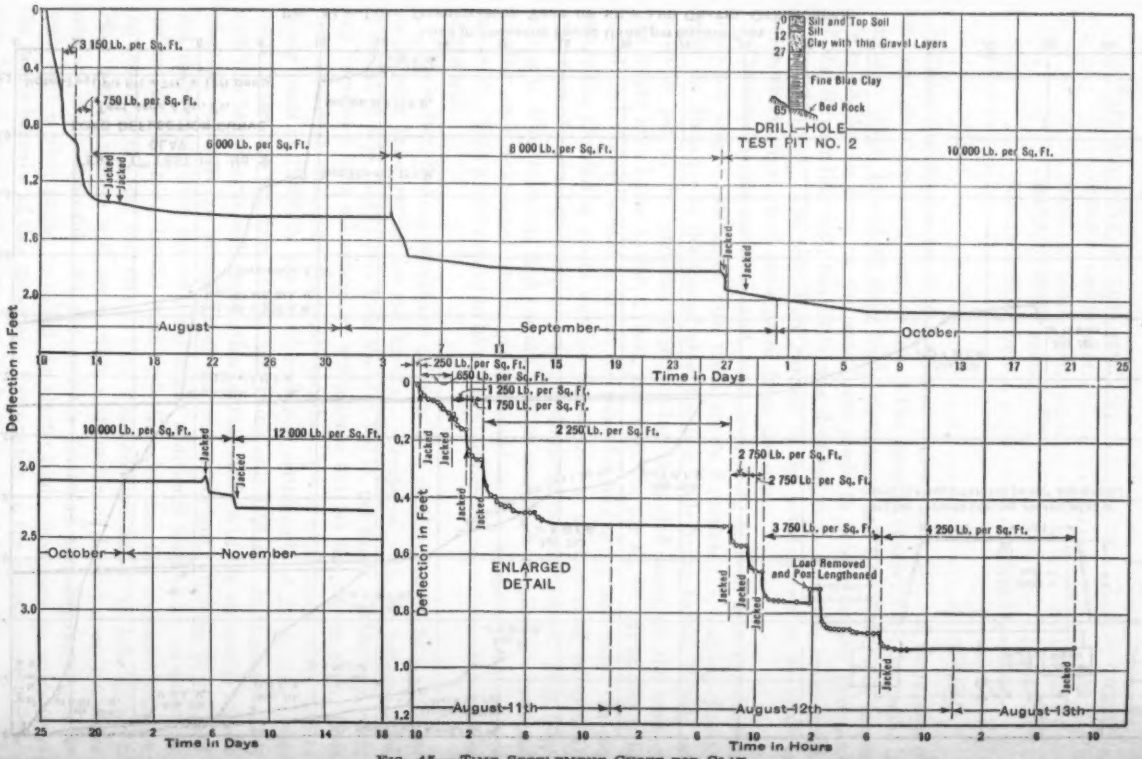


FIG. 45.—TIME SETTLEMENT CURVE FOR CLAY.

results with the actual settlements of the larger structure. It is thus hoped to have adequate data when the ultimate structure is designed.

The paper has been a provocative one, and the writer hopes that in contributing the results of his experiments, he will have helped in building up the much needed data. It is also hoped that the author will indicate any omissions in this description of the soil which would have made the data and tests more helpful to other engineers dealing with like material.

M. L. ENGER,* M. AM. SOC. C. E. (by letter).—The author has referred to tests made at the University of Illinois prior to 1916 on the transmission of pressure through sand. The results of tests made since that time, which were reported in part in the discussion of "Transmission of Pressure Through Solids and Soils and the Related Engineering Phenomena",† by George Paaswell, M. Am. Soc. C. E., are thought to be of sufficient interest to be given here in greater detail.

TABLE 10.—SUMMARY OF EXPERIMENTS TO DETERMINE DISTRIBUTION OF PRESSURE IN SAND BALLAST ARRANGED IN THE ORDER IN WHICH THE TESTS WERE MADE.

Series.	Depth of ballast, in inches.	Diameter of plates, in inches.	Maximum Applied Load.		Reaction percentage of applied load.
			In pounds.	In pounds per square inch.	
1c	38	21	6 700	19.2	41
1c	38	21	7 500	21.4	56
1a	38	36	13 650	13.3	57
2a	30	36	21 800	21.4	86
2b	30	30	15 500	21.8	85
2c	30	21	11 900	34.0	93
2d	30	13.5	4 150	28.8	77
3a	24	36	21 800	21.4	102
3a'	24	36	21 800	21.4	98
3b	24	30	19 600	27.6	98
3c	24	21	13 600	38.9	124
3d	24	13.5	6 700	46.5	116
4a	18	36	24 000	23.7	116
4b	18	30	21 800	30.7	130
4c	18	21	13 600	38.9	136
4d	18	13.5	6 700	46.5	150
5a	12	36	24 000	23.7	118
5b	12	30	11 900	16.7	132
5c	12	21	11 600	38.2	143
5d	12	13.5	5 400	37.5	125
5d'	12	13.5	5 400	37.5	112

The experiments were made on a pile of loose bank sand having a diameter at the base of 16 ft. At the greatest depth (38 in.) the sand pile had a top diameter of 6 ft. When the experiments on a given depth had been completed, 6 in. of sand were removed from the top of the pile to obtain the next depth of sand over the capsules. The base of the sand pile had a diameter of 16 ft. in all the tests, but the top diameter increased as the height of the sand pile diminished. The load was applied to the sand through circular plates, 13.5, 21, 30, and 36 in. in diameter, by a jack, and was determined

* Prof., Mechanics and Hydraulics, Univ. of Illinois, Urbana, Ill.

† *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 1563.

FIG. 45.—TIME SETTLEMENT CURVE FOR CLAY.

Time in Hours

Time in Days

by means of a calibrated steel spring. The arrangement of the apparatus is shown in Fig. 46.

The vertical pressures transmitted through the sand were measured by means of thirty-one pressure capsules placed in six radial lines in such a manner that there was one capsule directly below the center of the circular plate and three capsules on each circle of 3, 6, 9, 12, 15, 18, 21, 24, 27, and 30-in. radius about the center. The pressure capsules were the same as those used in the experiments reported in the Second Progress Report of the Special Committee on Stresses in Railroad Track.* They were calibrated before and after the experiments were made. The arrangement of the capsules is shown in Fig. 47.

TABLE 11.—VERTICAL UNIT PRESSURES AT VARIOUS DISTANCES FROM AXIS OF APPLIED LOAD ON CIRCULAR PLATES, EXPRESSED AS PERCENTAGES OF THE AVERAGE APPLIED UNIT PRESSURE.

Series.	Depth of ballast, in inches.	Diameter of plate, in inches.	Distance from Axis of Applied Load, in Inches.										
			0	3	6	9	12	15	18	21	24	27	30
1*a*	38	36	50	49	49	42	36	29	16	14	9	8	4
2*a*	30	36	74	73	78	68	53	44	32	21	13	6	6
3*a*	24	36	111	115	86	84	68	44	35	32	15	6	5
3*a*′	24	36	111	106	93	88	66	53	33	20	13	6	6
4*a*	18	36	160	150	132	110	80	61	31	17	10	5	6
5*a*	12	36	184	205	148	122	90	64	26	11	4	3	3
2*b*	30	30	64	64	64	53	38	32	20	13	8	5	2
3*b*	24	30	101	104	84	73	53	35	21	10	8	3	4
4*b*	18	30	136	154	114	91	64	37	15	9	4	3	2
5*b*	12	30	199	185	140	108	62	37	9	3	2	1	2
1*c*	38	21	18	17	16	11	10	6	3	1	1	0	0
1*c*′	38	21	18	16	17	17	11	12	6	3	2	2	1
2*c*	30	21	38	39	39	28	18	17	8	8	3	1	1
3*c*	24	21	72	80	63	46	24	18	6	5	3	1	3
4*c*	18	21	108	115	81	52	27	14	5	3	1	3	1
5*c*	12	21	189	141	104	64	24	10	1	0	0	0	0
2*d*	30	13.5	18	17	16	11	7	6	3	2	1	0	1
3*d*	24	13.5	37	37	28	18	9	7	2	2	1	0	0
4*d*	18	13.5	60	61	41	25	8	5	1	1	0	1	0
5*d*	12	13.5	109	102	51	15	4	1	0	0	0	0	0
1916 experiments.....	18	13.5	43	31	25	....	8	..	..	..	..	..	..
" "	12	13.5	100	64	40	....	5	..	..	..	..	..	..
" "	6	13.5	300	175	49	....	0	..	..	..	..	..	..

Table 10 shows the order in which the experiments were made and the maximum load applied in each test. The upward reaction computed from the readings of the capsules, expressed in percentage of the applied load, is also shown as a check on the accuracy of the measurements. It will be noted that the reactions are too small in the first three series of tests and too large in some of the later experiments. The error in the reaction is due to the fact that the average of readings at three points on each circle may not represent accurately the actual average pressure. Furthermore, the pressure on a capsule may be different from that which would be developed at that point if there were no capsule; that is, the pressure capsule itself has an effect on the intensity of pressure which it is to measure.

* *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 1409.

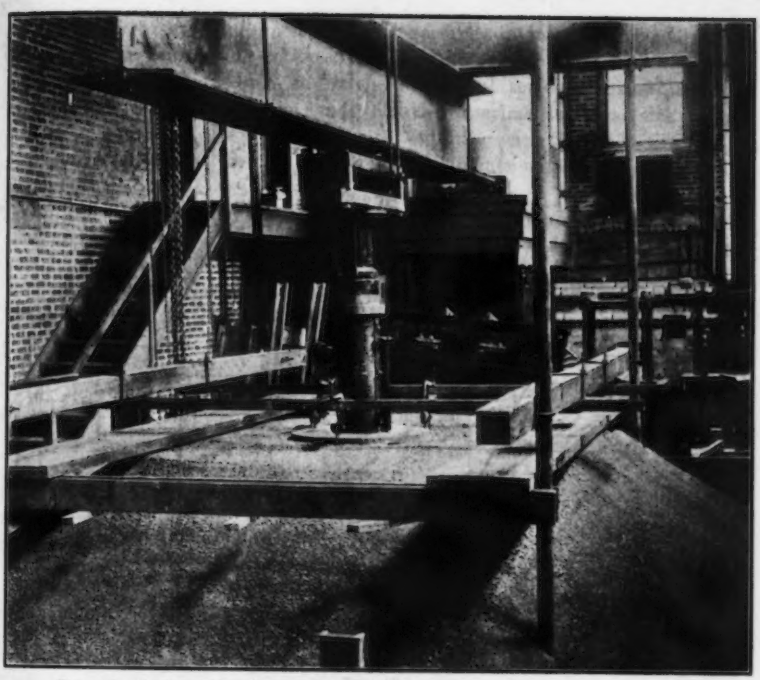


FIG. 46.—LABORATORY APPARATUS FOR TRANSMITTING LOADS THROUGH SAND.

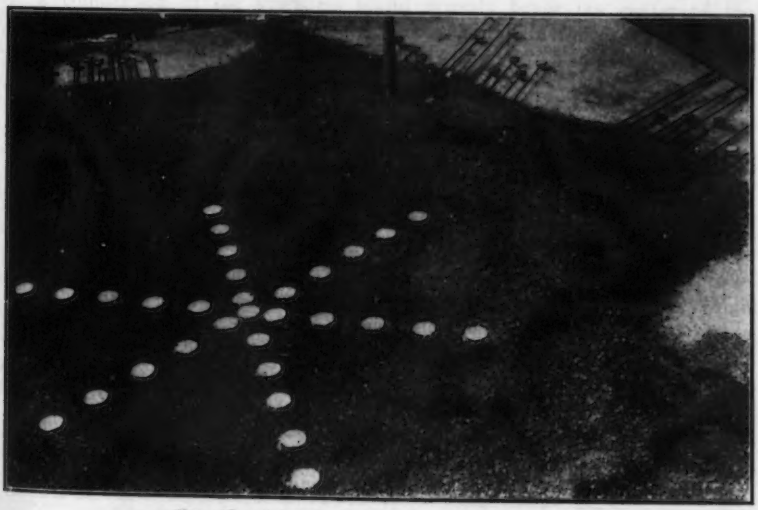
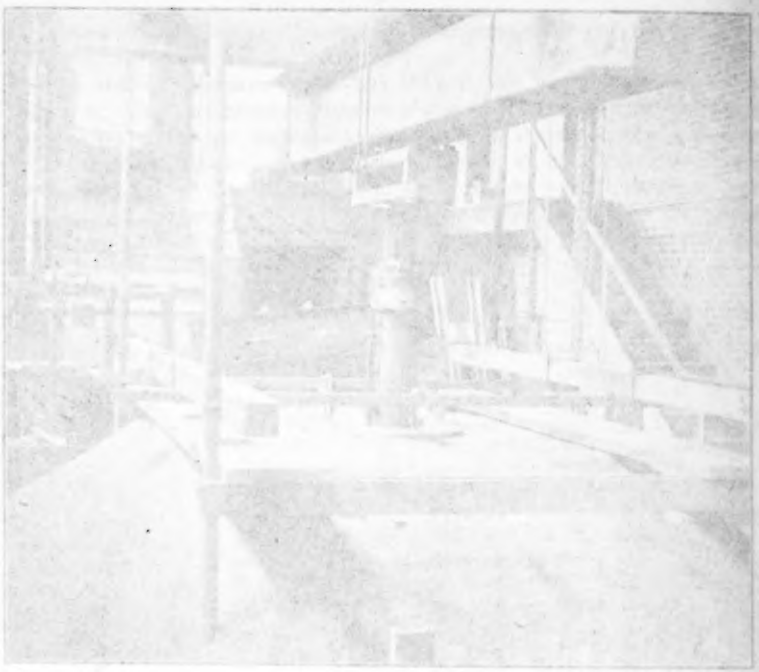


FIG. 47.—ARRANGEMENT OF PRESSURE CAPSULES.



The order of the experiments is of importance in interpreting the results because of the compacting action of the plates on the sand. This fact unfortunately was not foreseen, and in the first eight series of experiments no attempt was made to loosen the sand when the experiments on one size of plate had been completed. In the experiments beginning with Series 3*a* (Table 11), the sand was loosened before beginning experiments with a plate of another diameter.

A condensed summary of the results of the experiments is given in Table 11. The average intensity of pressure at various depths and distances from the axis of the applied load is given in percentage of the average intensity of load applied by the circular plate. It will be noted that there are many inconsistencies.

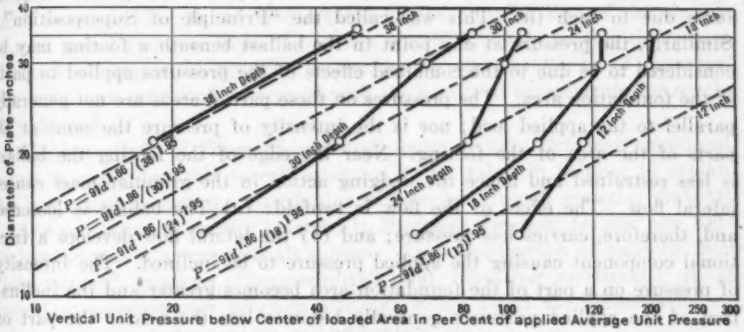


FIG. 48.

A comparison of the pressures found directly below the center of the loaded plates in these experiments with those found in the previous experiments at the University of Illinois is shown in Fig. 48. The dotted lines represent the equation,

$$P = 91 \frac{d^{1.86}}{h^{1.95}}$$

which was deduced from the earlier experiments. The agreement is not good. It was stated in the paper in which the equation was published* that, "the equation represents roughly the results of the experiments, but it is not probable that it has general application." The difference in the results of the two sets of experiments may be explained in part by the difference in procedure in making the tests. In the experiments reported in 1916 a very high intensity of pressure was applied. The plate was forced down several inches to the level used when taking readings. As a result the sand was compacted beneath the center of the plate and was loosened beneath the edge. Such a condition may exist under heavily loaded footings on sand. These heavy pressures were not used in the later experiments because of the fear that many of the capsules would be broken. It is thought that the distribution of pressure beneath circular footings used in practice will be betweeen the ex-

* *Engineering Record*, Vol. 73, No. 4, January 22, 1916, p. 106.

tremes represented by the tests reported in 1916 and the tests represented by Table 11.

The intensity of pressure at any point in the ballast beneath a footing is not a definite percentage of the average intensity of the applied load, but is different for different degrees of compactness of the ballast and for different intensities of applied pressure. It is, therefore, not possible to draw "pressure bulbs" which have general application. The high intensities in the tests at the University of Illinois before 1916 are only to be found when the plate is carrying nearly its ultimate load.

It was shown in the Second Progress Report of the Special Committee on Stresses in Railroad Track* that the intensity of pressure at any point in ballast beneath several loaded ties may be found by adding together the pressures due to each tie. This was called the "Principle of Superposition".† Similarly, the pressure at any point in the ballast beneath a footing may be considered to be due to the combined effects of the pressures applied to parts of the foundation area. The pressures on these partial areas are not generally parallel to the applied load; nor is the intensity of pressure the same at all parts of the area of the footing. Near the edge of the footing the ballast is less restrained and hence the wedging action in the granular mass causes lateral flow. The effect of the flow is twofold: (*a*) The ballast is loosened and, therefore, carries less pressure; and (*b*) the lateral flow develops a frictional component causing the applied pressure to be inclined. The intensity of pressure on a part of the foundation area becomes greater and the inclination of the applied pressures generally becomes less the nearer the part of the area is to the axis of the applied load. The flow of the granular mass away from the center of the plate is shown by heavy radial scratches on the bottom of the circular cast-iron plates used in some of the experiments. In the experiments made for the Special Committee on Stresses in Railroad Track*, heavily loaded railroad ties on rock ballast were sometimes torn in two by the frictional forces developed. The flow of the sand during a sudden failure of the ballast beneath a loaded plate is strikingly shown by illustrations given in that report.‡

The distribution of pressure in a granular mass is most easily comprehended by a study of "lines of pressure"§ which are analogous to lines of force in a magnetic field. If a load of, say, 100 lb. is applied to a granular mass, then on every horizontal plane below the loaded area there must be an added pressure of 100 lb. If the distribution of the vertical pressure is known on a number of horizontal planes at different depths, 100 diverging lines may be drawn, each of which represents 1 lb. of pressure. In regions of high intensity of pressure the lines are close to each other. If, for example, the pressure is 30 lb. on 1 sq. in. of one of the horizontal planes, then there will be 30 lines passing through that square inch. The lines spread as the distance down

* *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 1409.
† *Loc. cit.*, p. 1560.
‡ *Loc. cit.*, p. 1437, Fig. 9.
§ *Loc. cit.*, p. 1572.

ward increases. By subdividing a footing into small areas on which the direction and the intensity of pressure are nearly constant, a bundle of "lines of pressure" may be drawn for each of the areas and, by the principle of superposition, the intensity of pressure on any square inch of a horizontal plane may be found by counting the number of lines passing through that square inch and multiplying the number thus found by the number of pounds each line represents.

Lines of pressure for an 8-in. tie are illustrated in the Second Progress Report of the Special Committee on Stresses in Railroad Track.* The lines cross, indicating a concentration of pressure at a depth of 6 in. below the tie (found to be 163% of the average pressure by experiment). Below a depth of, say 6 in., the pressure in the ballast beneath the edge of the tie, and outward, is mostly due to the load applied by the opposite side of the tie. It may be stated that the lines of pressure should probably be slightly curved, although an empirical equation equivalent to straight "lines of pressure" gave results in very good agreement with the results of experiments for depths up to 24 in.†

The hypothesis of "lines of pressure" seems to have remained unnoticed, hence it seems desirable to call attention to its value in helping to explain the laws which govern the distribution of pressure through granular material.

The profession is greatly indebted to Professor Terzaghi for the new light he has thrown on a very difficult subject. The classification of clays by their "load-volume" relations, "permeability", and "shearing resistance", is an important advance over previous attempts which based the classification on color, grain size, water content, colloid content, etc.

The use of remixed material for determining "load-volume" relation and measuring "permeability" is open to the objection that the results do not apply to the undisturbed material. For example, in tests made on clay beneath the foundation of the new Materials Testing Laboratory at the University of Illinois, it was found that the water content of the undisturbed material was the same as that which corresponded to a pressure of more than 5 000 lb. per sq. ft. on the remixed material. The samples were taken at depths of about 3 and 6 ft. below the original surface where the pressures were, therefore, only a few hundred pounds per square foot. Both samples were taken below the water-table at the time of sampling, and the lower one was below the lowest water-table. Undisturbed clays probably have their flat, mica-like particles oriented in such a way that the voids are less than in the remixed material.

Methods should be devised for obtaining undisturbed samples for all the tests. This is very difficult even under the most favorable conditions and seems almost impossible in the case of material at great depths.

In the Red River Valley in Minnesota, North Dakota, and Canada, there is a widespread, deep bed of clay on which there have been many foundation failures. It has been found that buildings near railroad tracks have settled much more than similar buildings at a distance from the tracks, due to the

* *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 1577, Fig. 100.
† *Loc. cit.*, p. 1575, Table 19.

jarring action of heavily loaded trains increasing the instability of the clay mass.

Mr. Albert Jorgensen, who has conducted experiments in soil mechanics under the direction of the writer, makes the interesting suggestion that the distribution of pressure in clay beneath footings may be determined by experiments on models. A rather wet mixture of a uniform clay is to be placed in a large water-tight box with a thin layer of sand on the bottom and at the sides. A model of the particular footing to be tested is then to be placed and loaded. The surface of the clay is to be covered with water. The pressure on the footing will cause the water in the clay to flow away from the parts having the greatest intensity of pressure. When sufficient time has elapsed to reach the equilibrium condition, small samples are to be taken at various points in the mass beneath the footing and their water contents determined. From the "load-water-content" curve of the particular clay the water contents so determined may be translated into the intensities of pressure in the clay at the places from which the samples were taken. Measurements of the settlement of the footing, or of its parts, could also be taken. It is recognized that the box would have to be of large size to eliminate the effect of the boundaries on the distribution of pressure.

ALBERT JORGENSEN,* ESQ. (by letter).—This paper appeals to one who has studied it thoroughly because it shows that the author actually took some of the soil and determined its mechanical properties. He found that there is a great difference between the mechanical behavior of sands and clays and that clays vary greatly among themselves.

The capillary pressure of the water in the multitudinous, very small interstices in a clay is a factor of importance. It governs the volume occupied by a given clay under given conditions and under a given intrinsic pressure. This is one of the outstanding differences between sands and clays. The phrase, "under given conditions", is significant. The volume of a portion of clay at a given pressure (assuming that water is present) depends on the history of the clay.

If a sample of clay, in a condition near its lower liquid limit, is compressed by a certain pressure (p_1 lb. per sq. ft., for instance), it assumes a volume, definite for that clay, for the given pressure and the conditions of the test. Under the same pressure (p_1), but after having first been loaded to p_2 lb. per sq. ft., the same clay might occupy a different volume. For instance, if a sample of the same clay is made to sustain a pressure equal to p_1, but after having sustained a larger pressure (p_2), the volume will be smaller. Expressed differently, the pressure-moisture tests show that the volume at a given pressure depends on whether that pressure has been reached after positive compression only or after the clay has become re-saturated in recovering from a greater compression. The volume also depends on the intensity of the pressure attained before re-saturation was commenced and the number of times the clay has been compressed and re-saturated.

* Instr., Dept. of Gen. Eng. Drawing, Univ. of Illinois, Urbana, Ill.

The pressures may be applied mechanically in a manner designed to simulate actual conditions in the field; or they may be applied by the capillary pressure of the water which assumes enormous values if the sample is dried. Partly drying a sample would affect the clay to a degree comparable to that caused by a pressure which would produce the same change of volume.

Clay and mud beds, with the possible exception of recent delta and lacustrine deposits, have been subjected to unknown pressures and possibly may have been partly dried and re-saturated several times.

For these reasons a bed of clay below the level of ground-water is apt to be found dryer than could be explained by the existent pressure. For instance, the soil taken from beneath the location of a footing on the site of a certain building was found to be dryer even than the average unit pressure of the building would cause it to become. This soil was several feet below the level of ground-water. The pressure-moisture test, in this case at least, indicated that the soil had been compressed either by weight or capillary pressure.

The question then arises, as to what is the value of load-moisture tests in such cases. This would be answered if methods of sampling and technique for making tests on the soil in its original condition were at hand, but such methods do not seem to be forthcoming.

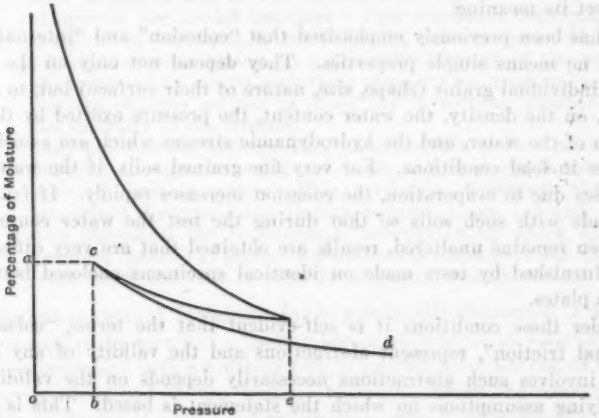


FIG. 49.

The next question which naturally arises is, "How may the tests be made on disturbed samples which will give a fair measurement of the condition of the undisturbed soil?"

The writer believes that a load-moisture test performed as follows, would give results which would aid greatly in formulating practical judgments concerning settlements due to change of volume of the soil. Wet some of the soil to a condition near its lower liquid limit and compress it in a cylinder by means of a piston. The soil should rest on a solid filter which will allow the water to pass to and from the soil as the pressure is varied. Compress the soil to such a point (Point *e*, Fig. 49) that when the pressure is reduced to *b* (that

pressure imposed on the soil in place), the moisture content will equal that of the soil in place (Point *a*). This would require nicety in testing and could probably be attained only after several trials. Then compress the soil to any desired degree.

The writer believes that the curve, *c d*, so obtained, would be accurate enough as a basis for practical judgments. It might have more value if obtained after several compressions and re-saturations.

In order to apply a pressure-moisture curve to aid in estimating settlement, it is necessary to know the distribution and intensity of pressures which exist under foundations. Tests by M. L. Enger, M. Am. Soc. C. E.,* and Messrs. H. E. Goldbeck,† and J. A. Moyer,‡ indicate that the pressures are far from uniform and may have values several times the average unit pressure.

CHARLES TERZAGHI,§ M. AM. SOC. C. E. (by letter).—This paper has called forth discussions from nineteen engineers in the United States, Russia, India, and China, representing a wide range of experience. While demonstrating the widespread interest in foundation problems, the discussion also disclosed the fact that few readers have had an opportunity to consult the publication from which the writer's Statement (*a*) was reproduced. For this reason, and on account of the condensed form of the statement, there was a tendency to misinterpret its meaning.

It has been previously emphasized that "cohesion" and "internal friction" are by no means simple properties. They depend not only on the character of the individual grains (shape, size, nature of their surface) but, to a variable degree, on the density, the water content, the pressure exerted by the surface tension of the water, and the hydrodynamic stresses which are associated with changes in load conditions. For very fine-grained soils, if the water content decreases due to evaporation, the cohesion increases rapidly. If friction tests are made with such soils so that during the test the water content of the specimen remains unaltered, results are obtained that are very different from those furnished by tests made on identical specimens enclosed between permeable plates.

Under these conditions it is self-evident that the terms, "cohesion" and "internal friction", represent abstractions and the validity of any statement which involves such abstractions necessarily depends on the validity of the simplifying assumptions on which the statement is based. This is obviously also true of the writer's Statements (*a*), (*b*), and (*c*). These refer to two different materials: (*a*) Those for which the bearing power depends essentially on cohesion, internal friction representing a negligible item; and (*b*) perfectly cohesionless materials. The validity of the statements is limited by the following assumptions:

(1) The shapes of the loaded areas are geometrically similar.
(2) The term, "settlement", refers strictly to the total downward movement measured after the structure has reached a state of complete rest.

* *Engineering Record*, Vol. 73, p. 106.
† *Proceedings*, Am. Soc. for Testing Materials, 1917.
‡ *Engineering Record*, May 30, 1914 and March 13, 1915.
§ Prof., Foundation Eng., Mass. Inst. Tech., Cambridge, Mass.

(3) The comparison between the settlement of loaded areas of different size should be confined to those produced by unit loads which are less than about one-half the ultimate bearing capacity for the smallest area.

(4) Hooke's law need not necessarily be accepted, but Poisson's ratio is assumed to be a constant.

Assumptions (1) to (4) apply to both Cases (*a*) and (*b*). The following additional assumptions apply to Case (*a*):

(5) The mechanical properties of the loaded material (relation between stress and strain and the compressive strength of the material) are practically independent of time and of the depth below the surface.

(6) At any point of the underground, failure occurs as soon as the difference between the greatest and the least principal stress exceeds the compressive strength of the material.

For Case (*b*), Assumptions (5) and (6) should be replaced by the following:

(7) The mechanical properties of the material (coefficient of internal friction) are practically independent of time and of the depth below the surface.

(8) The loaded material is incompressible; that is, the settlement of the load is merely due to the deformation, but not to the volume change of the loaded material. This condition combined with Assumption (4) leads to the following consequence: If all the principal stresses acting on a prismatic element of the material are increased by multiplying their intensity by equal amounts, the element changes neither in shape nor in volume, irrespective of the intensity of the stresses.

(9) At any point of the underground, failure occurs as soon as the ratio between the greatest and the least principal stress exceeds a definite limiting value. This condition represents the most important distinguishing feature between Cases (*a*) and (*b*); it also represents the fundamental assumption on which all earth pressure theories of perfectly cohesionless materials since Coulomb were based and may be considered almost strictly correct.

The subject content of Statement (*a*) is graphically illustrated in Fig. 50. Curve C represents the result of a loading test performed on an area (circular, square, or rectangular) with a width of 1 ft., supported by either one of the two ideal materials described by the writer. Curve C_1 shows the characteristic results of a loading test performed on an area with a geometrically similar shape, but with a width of 3 ft. supported by the first of the ideal materials; Curve C_2 applies to the second. The term, "ultimate bearing capacity", refers to the pressures, p_1 and p'_1, respectively, at which a very small increase in load produces a very important increase in settlement. The settlements produced by equal unit loads acting on areas of various sizes will be termed the "equivalent settlements" of those areas. Using these terms, the laws illustrated by Fig. 50 would be:

(*A*) If the ultimate bearing capacity of the loaded ideal material is essentially due to cohesion, the results of the loading tests performed on areas with a width, d, equal to 1 ft. and 3 ft., respectively, would be represented by Curves C and C_1 (Fig. 50). Under a load of p per unit of area, which is assumed to be

much smaller than p_1, the settlement, S_1, of the large area is approximately three times that of the smaller area, S; or, in other words, the equivalent settlement increases with increasing values of d. On the other hand, at a unit load of p_1, a small increase of the load produces for both areas a very large increase of the settlement, because both curves, C and C_1, bend steeply downward as soon as the load approaches p_1. Therefore, it is correct to state that the ultimate bearing capacity is independent of the size of the area.

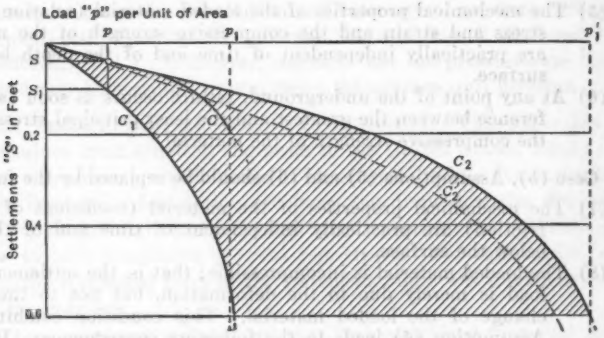


FIG. 50.—SETTLEMENT CURVES FOR LOADED AREAS.

(*B*) If the loaded ideal material is perfectly cohesionless, a similar pair of loading tests would be as plotted in Curves C and C_2 (Fig. 50). For a load of p per unit of area, the settlement, S, is practically independent of the size of the area, provided p is substantially smaller than p_1. On the other hand, the small area practically yields under a load of p_1 per unit of area, while the larger area must be loaded with $p'_1 = 3\,p_1$, in order to produce ultimate failure. Hence, it is claimed that the ultimate bearing capacity of the ideal, perfectly cohesionless materials (Law *B*) increases with the width of the area.

There was some doubt in the minds of most of the discussors as to whether Laws (*A*) and (*B*) were derived from theory or from experiments. To remove this doubt, it should be stated that the conclusions were obtained entirely by theory which was based on Assumptions (1) to (9), inclusive. As a matter of fact, if the validity of these assumptions is granted, Laws (*A*) and (*B*) need no proof, because they represent a simple consequence of the laws of similitude. Hence, any inconsistency between theory and practice must be ascribed to the properties of the material being at variance with those of the assumed, ideal materials.

If the properties of any soil conform reasonably well with the assumptions of theory, the laws represented by Fig. 50 must be valid. However, a comparison between the ideal materials and the actual soils leads to the conclusion that clean, cohesionless sands differ from the ideal cohesionless material inasmuch as they are not perfectly incompressible. If the intensity of all the stresses acting within the loaded material increases at the same rate, compression of the underground takes place which, in turn, causes a certain amount of

settlement in addition to that caused by distortion. Therefore, the empirical curve (C'_2 in Fig. 50) would be somewhat steeper than the theoretical curve, C_2; also, the ultimate bearing capacity of the loaded material may be slightly reduced for the same reason. However, since the compressibility of sands has already been thoroughly investigated, it is a simple matter of estimating roughly the discrepancies in every case that may come up in practice. For dense sands, the discrepancy between theory and practice is very small, while for loose sands it is apt to be important.

By comparing the ideal cohesive material represented by Curves C and C_1 (Fig. 50), with actual plastic uniform clays, the conclusion is reached that all assumptions are fairly accurate, except for a certain increase in the cohesion of the clay caused by progressive consolidation of the loaded material. Yet this increase could not possibly account for the striking discrepancy between the settlements which should be expected for clay foundations from the results of loading tests and the actual behavior of buildings standing on such materials. Among the fundamental assumptions of the theory, there is just one which could possibly be made responsible for the apparent defect of the theoretical results. According to Assumption (2), the term, "settlement", refers strictly to the total downward movement until the structure comes to rest. In practice, the settlement observations on buildings are made, at best, a few years after the buildings have been constructed, and when the settlement is still taking place.

The time which has to pass until the settlement of a fair-sized building resting on clay would approximately conform to Laws (A) and (B) obviously depends on the period required for the excess water to drain out of the clay beneath the building. This time, in turn, depends on the permeability of the clay and can be estimated by the theories previously published by the writer.* Suppose, for instance, that the settlement due to pure compression of the clay beneath an area 1 ft. wide was, after 5 days, 0.5 in. According to theory, for an area with a geometrically similar shape, 20 ft. wide, subjected to exactly the same load per unit of area, it would take $5 \times 20 = 100$ days to settle through the same distance of 0.5 in., provided the excess water could not escape except through the exposed surface of the clay, surrounding the loaded area; or, in other words, provided the bases of the loaded footings were water-tight. On the other hand, if the bases of the loaded footings were permeable, the settlement of both the small and the large areas, after 10 days, would be approximately identical. However, for the small area, the settlement would stop after it had reached during a period of several months a total of about 1.5 in., while for the larger one it would continue with decreasing speed until after several generations it would reach an ultimate value of $1.5 \times 20 = 30$ in.

Thus, it seems highly probable that the apparent contradiction between theory and practice for clay foundations is due not to a defect of the theory, but to the attempt to correlate the settlement of small loaded areas—measured almost as the footing comes to rest—with that of full-sized buildings in which the settlement has scarcely begun. As a matter of fact, in old cities with

* "Erdbaumechanik", Wien, 1925.

buildings resting on clay, or on very slightly permeable mud, such as parts of London or Constantinople, for example, there is ample evidence of progressive subsidence. The City of Chicago, Ill., too, despite its comparative youth, is crowded with instructive examples. A section of Cambridge, Mass., has subsided more than 2 ft. since 1880. If it were possible to compare the results of a loading test on clay with the settlement which a raft foundation, resting on the same clay, had undergone not 1 year but 200 years after the building was constructed, the agreement with theory would be much more satisfactory.

These statements are corroborated by the fact that the examples presented in the discussion as arguments against the validity of the writer's statement, referred entirely to clay foundations, on which the settlements are retarded by the enormous resistance of the material against the escape of the excess water. No contradiction was found to exist between theory and practice for foundations of friable limestone, muck soils, silt, and sand, and other fairly permeable materials (see, for instance, the discussions of Mr. White on limestone, Mr. Shaw on muck soils, and Mr. Wing on sand and gravel).

The bearing capacity of most of the more permeable soils commonly found in practice, for example, sands with traces of silt, or deposits of "muddy sands", is appreciably influenced by both cohesion and friction, the relative importance of cohesion depending on the composition and on the density of the material. For such soils, a loading test performed on an area 3 ft. wide should furnish a curve somewhere within the shaded area in Fig. 50, its shape and position depending on the degree to which cohesion participates in producing the bearing capacity.

Attention should also be called to the fact that Laws (*A*) and (*B*), or any rules for the interpretation of load test results, are based implicitly on the assumption that the soil is practically uniform and homogeneous to a depth at least equal to 1.5 times the width of the raft foundation, the settlements of which are to be predicted, or of the width of the area over which the footings are distributed.* Since in practice this condition seldom exists, the field for successful application of the load-test method for determining the admissible bearing value of the soil is far more limited than most engineers seem to realize.

In his paper the writer quoted the rules illustrated by Fig. 50 (Laws (*A*) and (*B*)), merely for the purpose of demonstrating that no real contradiction exists between the different settlement observations quoted in the paper (Chicago, San Francisco, or the test results of Goldbeck and of von Emperger), and that the apparent contradiction between these test results disappear if they are "examined in the light of applied mechanics". In all these cases, the properties of the loaded materials (hardpan in Chicago, succession of layers of clean or sticky sand in San Francisco), or the conditions under which the tests (by Goldbeck and von Emperger) were made, warranted the application of these principles (see Fig. 50). In contrast to this, no such application would be justified for correlating the results of a small-scale loading test on clay with the observations covering a period of a few months or years of the history

* According to Assumptions (5) and (7), p. 381, strict validity of the laws, requires uniformity of soil conditions to a depth equal to infinity.

of the settlements of a raft foundation resting on the same clay. In brief, the application of any rule to practical cases should be preceded by a careful investigation in order to determine to what extent, if any, the practical conditions agree with the fundamental assumptions of the rule. Due to the wide variation in soil properties no rules can be established that are valid for all soils.

The discussions deal essentially with two main topics—the *Engineering News* formula and the interpretation of the results of loading tests.

Among those who discussed the *Engineering News* formula, only one had anything to say in its favor. The others rejected it more or less emphatically, and without exception, on the basis of past experience. However, little attention was paid to what the writer considers by far the most important feature of the situation, namely, that the most general pile-driving formula (Equation (3)), is fundamentally rigorous, provided one grants the validity of the basic assumptions. The most important of these assumptions concerns the identity of the static and the dynamic pile-driving resistance. The failure of the formula to give satisfactory results for pile-driving through clay or saturated silts is due merely to the fact that, in these cases, the conditions existing in practice fail to conform with the theoretical assumptions.

These remarks lead to the attitude of the discussors and of engineers in general toward the writer's statements as illustrated in Fig. 50. Some experienced engineers have accepted these statements as not being in contradiction with their own observations. Others, also, backed by broad personal experience, have rejected them. This fact should arouse the suspicion that conditions are not simple enough so that solutions can be found in the shape of rough and ready half empirical formulas. In a third group of discussions, attempts have been made to disprove the statements by proposing formulas which are claimed to agree better with practice.

These reminded the writer of the many discussions he has read about pile-driving formulas in past years. The method of approaching the problem was always the same. As often as a new formula appeared it was found invariably that the results furnished by it did not agree with many of the observed facts; therefore, it was argued that the formula must be modified. Nobody seemed to suspect that its deficiency might be due to the nature of an assumption in common to both the criticized and the proposed formulas. Hence, the discussion of the pile-driving formula revolved for half a century in a vicious circle.

None of the discussors seemed to realize that the writer's statements stand as firmly "on their feet" as applied mechanics, provided the fundamental assumptions conform with the conditions existing in practice. The assumptions cover all the essential mechanical properties of the soils, except those which depend on time as, for instance, the compression of a clay or of a very fine saturated silt. In a similar manner, the assumptions on which Equation (3) is based consider all the essential elements of pile-driving, except the fact that, in many cases, the dynamic and static pile-driving resistance are not identical. Hence, it is justifiable to conclude that Laws (*A*) and (*B*) must be correct, except for cases in which the time factor partly or wholly upsets their validity for the brief period covered by the observations.

Instead of presenting this simple, but very important argument, the members of the third group of discussors merely tried to replace the laws, which have a limited but known range of validity, by others which are neither justified by rigorous analysis nor supported by a known range of validity. In doing so, this group repeated the mistake made by discussors who tried to replace the classical pile-driving Formula (3) by other more objectionable ones, instead of inquiring as to the fundamental cause of the observed disagreements.

These remarks also apply to discussors of the second group who attempted to question the soundness of the writer's statements solely on the ground of past experience. After realizing the limitations of theory, these engineers may ask: What then is the use for establishing rules at all? The answer to this question touches the most vital point of the science of foundations, and it cannot possibly be over-emphasized.

Settlements, as encountered in practice, depend as a rule on a number of circumstances, such as depth of foundation; presence of other structures in the vicinity of the new building; the occurrence of alternate layers of slightly permeable and markedly permeable strata; total thickness of the compressible deposits; relative position of the weakest spots of the underground with reference to the base of the foundation; and many others. In brief, the settlement of a structure is a function of so many variables that it would be hopeless to arrive even at crude and empirical conceptions of the causes of settlements unless systematic efforts are made to evaluate the relative importance of each one of the individual variables. This can be done merely by establishing a theory, not of the complex materials encountered in practice, but of a set of ideal materials with certain very simple properties, each one of which will serve as a means of studying one individual aspect of the real soils.

By such systematic analysis only can engineers gradually gain a deeper insight into the workings of actual soils. Since by necessity the very best analysis of that type falls far short of taking into account all the conditions that exist in practice, there always will be a certain gap between theory and practice, which can be bridged merely by experience. Considering the existence of this gap, the writer agrees heartily with Mr. Gow's warning against readily accepting common formulas for use in actual practice. The writer even doubts whether formulas with a fairly general degree of validity could be established at all. Theory in connection with foundation engineering should be regarded merely as an essential supplement to experience, to be used as a tool for acquiring a deeper insight into the causes of observed foundation defects and for evaluating the relative importance of the factors in existence.

Without a more profound analysis of phenomena encountered in practice, experience is doomed to remain the same accumulation of incoherent, meaningless facts which fill the space between the covers of many textbooks on foundation engineering. The time may come when a man with experience, but without a thorough theoretical training, will be considered almost as unfit to be trusted with a design of an important foundation as the theoretician who has had no experience. In structural engineering, this time came nearly a century ago.

The writer hesitates to accept Mr. Gow's advice concerning the *Engineering News* formula. If, as Mr. Gow states, it is a fallacy to rely too strongly on the results of individual pile tests, the application of the *Engineering News* formula seems to represent an even greater fallacy.

It is significant that Mr. Goodrich abandoned the formula long before the writer had even heard about its existence. The observations made by Mr. Goodrich concerning the stress distribution over a square bearing plate should be an incentive for similar investigations of raft foundations resting on clay soils by means of Goldbeck pressure cells or similar devices.

The experience reported by Mr. Endersby reminds the writer of a similar incident which occurred in Galveston, Tex. On the south side of the Island of Galveston, to a depth of about 50 ft., the underground consists of very fine sand mixed with powdered shells. Because of the very dense structure of this sand, it is considered impossible to drive piles to a depth of more than 15 or 20 ft. without smashing the pile-head. According to Colonel Schley, Government Engineer of Galveston, some piles driven with a steam hammer to refusal settled afterward several inches under a static load of less than 15 tons. This and the incident mentioned by Mr. Endersby are obviously decompression phenomena which can also be reproduced in the laboratory. If a layer of sand that is laterally confined is subjected to a rapidly applied pressure up to a certain value and then the pressure is kept constant (possibly by a testing machine), the counter-pressure of the sand against the piston first drops rapidly and then slowly to less than one-half the original reaction. In connection with this statement, it should be emphasized that there seem to be no records of conspicuous decompression phenomena except for cases in which the pile-driving was done with a steam hammer that could apply pressure very rapidly. The intensity of the decompression phenomena may be considerably increased by the presence of a layer or pocket of very loose sand a short distance beneath the point of the pile. Since the number of known conspicuous decompression phenomena on piles is exceedingly small compared to the number of cases in which the piles behaved normally, such a possibility could at least be considered.

Mr. Endersby's statement that the result of the tests with three different hammers was "a good test for consistency of the [*Engineering News*] formula as applied to various types of hammers", should be accepted with caution. The satisfactory agreement between the results appears to be mere chance; otherwise, it would always exist. To mention one example among several on record: When driving the piles for the Mariotis Lake Bridge, in Egypt, three different pile-drivers were used: A steam hammer (weight, $R = 4\,400$ lb.; drop, $h = 3.28$ ft.); a hand-operated drop hammer ($R = 1\,320$ lb.; $h = 6.56$ ft.); and a steam drop hammer ($R = 4\,400$ lb.; $h = 3.28$ ft.). The points of all the piles penetrated the same stratum of sand.* The *Engineering News* formula furnished for the safe bearing capacity of these piles the average values of 66 800, 16 800, and 27 600 lb., respectively. If the *Engineering News* formula would furnish consistent results for piles driven into the same stratum with

* According to Mr. W. Stross, in *Beton und Eisen*, 1908, p. 113.

different types of hammers, these three values would have been fairly identical. Mr. Endersby's statements show how a formula can acquire a good reputation by an act of Providence rather than by inherent merits. Since engineers very seldom have a chance to watch different types of pile-drivers operating simultaneously on a very homogeneous material, they are apt to base their opinions on an inadequately small amount of actual evidence.

Mr. Paaswell justly emphasizes the fact that the New York Subway "is unquestionably the world's best laboratory for the study and illustration of all types of foundation problems". The same was true for the Panama Canal in the field of stability of earth slopes. Unfortunately, very few results of lasting value were derived from the Panama experiment because of lack of funds available for purely scientific research. In New York, the unique opportunity for systematic research seems by no means to have been utilized to full advantage. The New York Subway remains a beautiful laboratory without equipment and without personnel. The profession should appreciate all the more the idealism of those few engineers engaged in subway construction who, by their own initiative, seriously endeavor to broaden technical knowledge on this subject by systematic experimentation.

Mr. Williams presents a theory based on three assumptions: (*a*) That settlement is merely due to compression of the underground; (*b*) that the total depth of soil compressed is limited by the depth at which the pressure intensity (per unit of area) drops below a certain limiting value; and (*c*), that the coefficient of compressibility, C, is independent of depth. Attention should be called to the fact that Assumption (*c*) could merely be considered approximately valid for materials with strong cohesion; for cohesionless materials, on conclusive, experimental evidence, it does not apply. Mr. Williams then computes the settlement of square areas with different widths, b. From Assumptions (*a*) to (*c*), it is evident that h (Fig. 18), increases in simple proportion with b. This fact is also expressed by Mr. Williams' formula, $h = \frac{b}{r}\left(\sqrt{\frac{w}{p}} - 1\right)$. By introducing the value of h into the terms preceding this formula, the same results are obtained as those expressed by Curve C_1, Fig 50. However, instead of retaining h, Mr. Williams ceases to consider it as a function of the width, b, without attempting to justify his step, thus transforming it into a constant. This operation leads to the formula, $W = A\,p + L\,f$, which contradicts his theory. Since his final formula does not represent the result of the theory, it could be accepted merely as an empirical equation and the writer considers that the empirical evidence is too fragmentary to warrant such a step.

The example quoted by Mr. Williams (the office building of the Michigan Central Railway Company in Detroit, Mich.) does not invalidate Curve C_1, Fig. 50. To conform with the assumptions of theory, the underground should be uniform to a depth equal to at least one and one-half times the width of the building. In contrast to this, from his own experience, the writer can state that the clay deposit of Detroit is far from being homogeneous. It consists of an irregular succession of layers of soft and stiff clay.

Furthermore, the observations quoted by Mr. Williams refer to a recently constructed building and settlements are not comparable as long as one of them still continues. For a building resting on clay foundations, the settlement twenty years after it was constructed is certainly not identical with that just after the building was completed. It may be many times more important. Thus, for instance, in Massachusetts, a short time after construction, the settlement of an important structure founded on plastic clay ranged between 0.1 and 1.1 in. One year later, the extreme settlements had increased to 0.6 and 2.8 in., respectively, and ten years later, they amounted to 1.0 in. and 7.6 in., respectively. In Chicago, conditions are not very much better.

Mr. Williams suggests reducing the discrepancy between the results of the loading test and the actual settlement by loading the soil around the test footing. To follow this procedure would mean an attempt to compensate for the effect of the time factor, which is absent, by adding a load factor. The importance of the time factor on the relation between settlements of small and large footings is exceedingly different even for the same clay, depending on the water content. Hence, it seems rather difficult to establish a reliable rule for computing the intensity of the compensating load which should be applied around the footing. Instead of trying to reduce the discrepancy between test results and the settlement of the proposed structure by introducing into his arguments one more element of uncertainty, the writer takes it for granted that this discrepancy (together with the possible effects of the lack of uniformity of the underground) represents inevitable defects of the load test method. Hence, when attempting to evaluate the probable settlement of a proposed structure, he bases his judgment not only on test results and his knowledge of soil physics, but also on past experience and on what the test borings show about the nature of the deeper strata. Test results and physical knowledge are used merely for establishing connection between past experience and the new situation. In no case would he dare to rely solely on a test result.

Mr. Williams states that the writer neglected the effect of the deflection of the foundation on the pressure distribution over the base of the foundation. For foundations on well compacted sand, the pressure-relieving effect of the deflections is very important indeed, but for soft soils, this is insignificant. This fact has been known for more than forty years. Since most raft foundations are supported by soft soils, designers do not often have an opportunity to take advantage of the pressure-relieving effect of deflection. The question concerning the distribution of pressures over the base of raft foundations is still in a controversial state, and the discussion by Mr. White of the same topic seems to indicate that engineers are by no means unanimous in assuming uniform pressure distribution to be safe.

Dr. Chatley calls attention to the slowness with which hydraulic fills, consisting of fine alluvium, expel their excess water. During 1928, under the writer's supervision, Glennon Gilboy, Jun. Am. Soc. C. E., made an exhaustive study of the process of consolidation of an hydraulic-fill dam. For this purpose, a shaft was sunk through the core of the dam and undisturbed samples were secured at various depths below the crest. The results

of these investigations represent an important contribution toward present knowledge of the consolidation process; they conform with Dr. Chatley's observations and with the conclusions previously drawn by the writer from theory.

Dr. Chatley's conception of the part that the molecular bond plays in the physics of granular masses is fundamentally sound, although it is still doubtful whether the molecules of two adjoining particles interact directly or whether the interaction merely takes place between the molecules of the skins of adsorped materials surrounding the particles.* The possibility undoubtedly exists that the degree of dispersion of a highly colloidal soil may gradually change. To date, very little is known about this process. The writer is inclined to believe that it is often associated with chemical changes and that it plays a very important part in Nature.

The most promising methods for the design of sheet-piling seem to be those based on the assumption of a circular sliding surface. These methods were developed by K. E. Petterson, Sven Hultin, and W. Fellenius, in Sweden.†

In 1920 the writer was at least as enthusiastic about colloids and about the possible benefits to be derived from colloidal chemistry for foundation and earth-work engineering as Mr. Holmes is to-day. However, during the succeeding years of struggle with the complex material called soil, this enthusiasm has dwindled to practically nothing. Some of the reasons of the progressive disappointment can be explained by means of Mr. Holmes' own data. He admits that "extracting the colloids from a soil to determine the amount of colloidal material is a long and laborious process". Therefore, he proposes a simplified method of his own. According to Fig. 20, an ignition loss of 3% may mean anything between 6% and 29% of colloid content.

Furthermore, since the physical properties of soils with equal "colloid content" are apt to be widely different, Fig. 20 is certainly not a very convincing argument. The following data from tests on a soil in Spanish Honduras may serve as an illustration. The soil had a clay content of 70% and, in its natural state, a water content of 33 per cent. The coefficient of permeability of this soil was found to be: (*a*) Undisturbed sample under primeval forest with 33% of water, $k = 2.6$ cm. per min.; (*b*) the same soil dried, powdered, and compacted, $k = 0.0024$ cm. per min.; (*c*) the same soil with the same water content as in Case (*a*), but after completely destroying its original structure by kneading, $k = 0.00000085$ cm. per min. In addition, by increasing the water content of Sample (*c*), from 20% to 40% the coefficient of permeability was increased from 0.00000010 to 0.00000250.

Hence, even if a rapid and reliable method should be found for determining the percentage of colloids, it may have but little practical value. The quality of a dam construction material depends essentially on its permeability, cohesion, and internal friction. No simple and universally valid relation exists between the colloid percentage and any one of these important physical properties. The situation becomes still more involved considering that

* "Mechanics of Adsorption and the Swelling of Gels," by Charles Terzaghi, M. Am. Soc. C. E., Fourth National Colloid Symposium, 1926.

† "Erdstatische Aufgaben," W. Fellenius, Berlin, 1927; "Erddruck, Erdwiderstand und Tragfähigkeit des Baugrundes," H. Krey, Berlin, 1926.

both the permeability and the cohesion of any soil may vary between extremely wide limits, depending only on the history and not on the substance of the soil deposit. Nothing except a refined chemical analysis could disclose the nature of the admixtures. Finally, some of the most excellent dam construction materials that the writer has ever encountered, which were splendidly fit for constructing earth dams without a core-wall, contained almost no colloids.

Considering these and many other facts, the writer cannot be blamed for his skeptical attitude toward those who hope to derive important practical benefits from colloidal research. He prefers to obtain data directly from the results of appropriate experiments. Nevertheless, any advance in the field of colloid chemistry is of outstanding interest to every one dealing with soils and with earthwork engineering. This interest, however, is at present of a purely scientific nature.

Mr. Feld thinks that the writer's effort to establish a more accurate interpretation of actual construction experience is a desire to "suddenly substitute a new 'science of foundations'." for precedent and experience. No such intention ever entered the writer's mind. The need is, not for a latent state of war, but a more intimate co-operation between field and laboratory. In establishing such co-operation, the absence of any authoritative nomenclature certainly is a serious handicap; but it is by no means the principal stumbling block. Far more detrimental is the widespread lack of training in abstract reasoning and the tendency to reject theory completely unless it leads, overnight, to conclusive results.

The reason the writer neglected to deal with the effect of the shape of the area on settlement is obvious. At the time when the paper was written, he did not know anything about it. Even in the case of the bearing of solid against solid within the elastic limits, the shape of the area has a very important effect, but all efforts made to solve the problem theoretically thus far have failed. However, since January, 1927, experiments have been conducted at the Massachusetts Institute of Technology, for the purpose of getting at least preliminary information on this controversial subject. The results obtained are interesting enough. They suggest that the effect of the shape of the area on both the equivalent settlement and on the ultimate bearing capacity may be very different, depending on the degree of cohesion.

For example, assume two loaded areas, one with a circular shape, of which the diameter is d, and the area, f; and the other one rectangular with the width, d, and a length several times larger than d, so that the area is F. For a material with dominating cohesion (Plasticine), the equivalent settlement for f was found to be smaller than that for F. On the other hand, the ultimate bearing capacity for f was found to be considerably greater than that for F. For perfectly cohesionless materials, the opposite relation was found to exist. The tests were repeated several times, and the results were remarkably consistent. Here, again, it seems a fact that the influence of the shape of the area may be very different, depending on whether the bearing capacity of the material is essentially due to cohesion or to internal friction.

The "sympathetic" settlement of adjoining footings may have very different causes. In the case illustrated by the writer's Fig. 11, a fence post

located at a distance of about 300 ft. from Building (*b*), had settled several inches. In this case, the sympathetic settlement was due to the presence of a layer of sand beneath a bed of unconsolidated clay. Another type of sympathetic settlement (settlement of the surface of the ground in the vicinity of a concentrated load) has been treated theoretically by the late Professor A. Föppl in "Vorlesungen über Technische Mechanik", which also contains the results of the tests performed in order to check the theory.

Mr. Housel presents a series of very interesting and valuable test results. However, from these results, he feels justified in concluding that the statements presented by the writer concerning the relation between the bearing capacity and the diameter of round plates, is false, and that they lead to absurd results. A similar opinion is known to be held by many other engineers.* The writer, therefore, may be justified in examining the arguments presented by Mr. Housel in somewhat greater detail.

At the outset of this closing discussion, the writer quoted the assumptions on which Fig. 50 was based. It also was shown that Laws (*A*) and (*B*), derived from these assumptions, are as unquestionable as the laws of applied mechanics, provided the assumptions agree with conditions existing in Nature. Therefore, if a discrepancy is found between theory and practice, it must be due to some factor not taken care of by theory. In the case of foundations resting on clay, an apparent discrepancy was found to be caused by the low permeability of the material which retards the settlement of a large footing much more than that of a small bearing-block. The advantages to be derived from such knowledge are manifold.

The first lesson learned is the intimate relationship that exists between the low permeability of the underground and the progressive settlement of the buildings which it supports. Furthermore, it is important to understand that a settlement of identical foundations on identical clays may be very different, depending on whether or not the clay contains streaks or layers of permeable sand through which part of the excess water of the clay may freely drain. It is also important to prevent the mistake of applying to a fairly permeable system of strata the experience previously obtained with foundations on very slightly permeable materials, such as plastic, homogeneous clays. That is what the writer calls a "scientific approach" to a soil problem; an attempt to uncover the responsible factors by isolating the variables and systematically determining their relative importance.

However, Mr. Housel selected another and apparently simpler method of dealing with his subject. Instead of attempting to investigate whether or not the statements quoted by the writer were consistent with the accepted laws of mechanics, he introduced into his discussion rather vaguely defined conceptions such as "shearing resistance" per unit per length of perimeter and "pressure bulb". These conceptions certainly are convenient for a general description of the conditions which exist in the underground of a loaded area; but, if it comes to rigorous analysis, they cease to exist. In applied mechanics, the term, "shearing resistance", refers exclusively to a unit of area, not to a unit

* *Engineering News-Record*, March 29, 1928, p. 520.

of length, and the pressure bulb is not referred to at all, except inasmuch as it indicates vaguely what is rigorously expressed by the formulas of Hertz and Boussinesq.

Considering these facts, it is very difficult to translate into terms of applied mechanics such statements as "the important consideration * * * is the effectiveness with which the different shaped plates adjust themselves to the pressure bulb."

Speaking of his test results, Mr. Housel admits that the results agree with conditions as defined by Curves C and C_1, Fig. 50. Yet, he rejects them because "* * * when the principle is applied to the much larger areas used in practice that the fallacy of this assumption becomes apparent". Then, without preceding analysis, Mr. Housel supersedes the term he rejects by a term of his own conception (Equation (11)), and simply claims that his term represents a satisfactory solution, valid for all soils irrespective of their physical properties. Nowhere in his discussion does he hint at the possibility that a difference in drainage conditions may have a far greater influence on settlements than a difference in compressibility or in internal friction.

Mr. Bailey touches the human side of foundation engineering by quoting two cases of failure which occurred in spite of thorough preliminary work. The question arises: Will it ever be possible to avoid entirely such tragic incidents? The writer does not believe so. At the same time, from his personal experience with foundation failures, he is convinced that more than 90% of the accidents which occur nowadays could be avoided by proper application of the results of soil research to the design of foundations. The others will remain inevitable, very likely forever, on account of the fragmentary character of the information furnished by test borings. By transforming this fragmentary information into complete geological profiles, the engineer automatically introduces into his forecast a personal element and with it an element of uncertainty. Thus, on construction work with which the writer was connected, careful test borings were made and all of them indicated the presence of a continuous bed of very stiff clay extending to a considerable depth below the base of the proposed raft foundation. Immediately before the reinforcement for the concrete slab was placed, water broke through the bottom of the pit, carrying large quantities of very fine sand. Subsequent investigation by supplementary boring showed that there was a narrow belt of quicksand enclosed within the clay bed at a very shallow depth below the bottom of the excavation pit and by mere chance the test borings missed it.

The risk caused by the fragmentary character of test-boring records will always continue to exist. In connection with dam foundations this inevitable risk is very much greater and very much more serious. The ultimate aim of the science of foundations merely consists in reducing the present intolerably wide margin between the factor of safety of the most and of the least solid foundations.

Mr. Condron describes an instructive case of foundation experience in connection with the Chicago blue clay. According to the test borings, the stiffness or the cohesion of this clay increased considerably with the depth.

Since the cohesion of the clay varied with the depth, and since, in addition, the clay represents a very slightly permeable material, Law (*A*) is not directly applicable to Mr. Condron's case, and the interpretation of the test results represents a rather complicated problem of soil physics.

Mr. Condron's method of handling the problem can be considered a typical example of a very conscientious and conservative approach along the lines of current practice. In order to determine wherein this practice may fall short of what is desirable, engineers may ask the question: What arguments led Mr. Condron to conclude that 3 000 lb. per sq. ft. is a safe load for the soil which he tested?

His most important argument undoubtedly was that the neighboring buildings, with foundations designed for a soil pressure of 4 000 lb. per sq. ft., showed no evidence of unequal settlement. After Mr. Condron had verified this fact, he made a loading test, with the soil around the loaded area remaining in place. From the favorable outcome of this test, plus the evidence furnished by the neighboring buildings, he reached his conclusion.

If the procedure for determining the admissible soil pressure is so simple, then why does the engineer need any of the more refined soil studies? To answer this rather vital question, suppose that an engineer has to construct a duplicate of Mr. Condron's building. Before preparing the plans for the foundation, he makes exactly the same test as Mr. Condron did, except to assume that there are no neighboring buildings the behavior of which would inform him about the quality of the underground. Furthermore, when making his test borings, suppose that the engineer discovers that, to a depth of 10 ft. below the base of the proposed footings, the underground is the same as that found by Mr. Condron. Below that depth conditions differ in one of three possible ways, as follows:

(*a*) Instead of homogeneous stiff blue clay, there is a homogeneous soft blue clay.

(*b*) Instead of homogeneous stiff blue clay, there is a stiff blue clay which contains several layers of dense but very permeable, water-bearing sand.

(*c*) Instead of homogeneous stiff blue clay, there is a "varved" clay, consisting of a succession of almost impermeable and fairly permeable layers about 1 in. thick.

Since, at a depth of 10 ft. below its base, the soil pressure exerted by the loaded 6-ft. square test footing is already negligible, the outcome of the loading test will be almost exactly identical with that obtained by Mr. Condron. Yet, the major part of the pressure bulbs of the full-sized footings will be in a material altogether different from those in the case cited by Mr. Condron. As a consequence, the ratio between the settlement of the test footing and of the full-sized footings will also be altogether different.

In Case (*a*), the settlement due to lateral yield of the soft clay will be much more important than it was when the deeper part of the stratum consisted of stiff material. In Case (*b*), the excess water in the clay beneath the full-sized footings can freely drain into the sand out of the clay, while

through the homogeneous stiff clay beneath Mr. Condron's building, it could not percolate except very slowly. In Case (*c*), the clay can discharge its excess water still more freely, and the settlements due to gradual consolidation are apt to be very important although the material gives the impression of being very stiff. Whenever, in practice, the writer encounters "varved clay", he knows there is trouble ahead, and experience has invariably confirmed his anticipation.

The engineer who pays little or no attention to the manifold characteristics of soils would be apt to trust the outcome of the loading test, and would consider Cases (*a*) to (*c*) to be identical with the one described by Mr. Condron. Hence, he would also consider 3 000 lb. per sq. ft. a safe load and to his surprise the result of his decision would be an excessive settlement of the structure. The task of developing a "science of foundations" consists in preventing such errors by painstaking analysis of the relative weight of each individual factor which could possibly influence the behavior of the proposed structure. Without such analysis, the profession would have always as many different rules for the interpretation of the results of loading tests as there are engineers who keep a record of their observations.

Mr. White's experience, derived mostly from construction work in New York, covers some of the most difficult fields of foundation engineering and, whenever he chooses to disagree with any of the writer's statements, the writer always feels compelled to "think it over twice" before he replies. The two points wherein Mr. White's opinions seem to differ from those of the writer are the value of loading tests and the relation between the bearing capacity and the density of soils.

The principal arguments against placing too much confidence in loading tests were explained in general at the beginning of this closing discussion and by means of an example in the writer's comment on Mr. Condron's discussion. The writer uses loading tests merely as a means of investigating the character of individual strata, but he never would dare to pass judgment on the probable settlements of a full-sized structure without careful consideration, not only of the results of the loading tests, but also of the influence of the characteristics of those strata between the elevation of the base of the pressure bulb of the test footing and that of the full-sized footing. The cases in which the soil is of a perfectly uniform character to a depth at least equal to the width of the building are very rare.

The relative density of a soil or of a rock is undoubtedly a factor of importance, but unfortunately the relation between the behavior of this soil and its density are by no means simple enough to furnish reliable information on the bearing capacity of the material. Thus, when experimenting with clays, the writer had an opportunity to compare two blue clays from two different localities. One of these clays, with a water content of 35% (volume of voids, 48%) was almost liquid, while the other with exactly the same water content and the same volume of voids was found to be stiff and had a very appreciable consistency. Similar although less conspicuous variations were found to exist when experimenting with sands.

This leads to a discussion of the effect of the shape of the grains. Mr. White is inclined to assume that the large voids ratio may have the same effect on the mechanical properties of a sand as the mica content. As a matter of fact, the compressibility of the same sand increases rapidly with its volume of voids. There is, however, one fundamental difference between pure sand and sand-mica mixtures which cannot be explained by Mr. White's assumption. This difference concerns the elasticity of the two materials as illustrated by the shape of the rebound curves. Thus, when experimenting with very fine quartz dust, it was found that the compression curve of this material was almost as steep as that of a sand-mica mixture, but the rebound curve was almost as flat as that of an ordinary sand, while the rebound curves of the equivalent sand-mica mixture rose very sharply. Since the rebound curves of the clays are as steep as those of sand-mica mixtures, the writer concluded that the clays and silts owe their mechanical properties essentially to the presence in the material of a high percentage of scale-like particles. At the time when this statement was published, it was presented as a hypothesis. Since then, Professor Goldschmidt, in Oslo, Norway, has succeeded in measuring the actual quantity of scale-like particles present in typical Norwegian clays,* and the data which he obtained checked so well with those derived from the sand-mica experiments performed at Massachusetts Institute of Technology that there is little doubt about the hypothesis being fundamentally sound.

From a practical point of view, the principal difference between very fine sand and an equally fine sand-mica mixture is that the volume of voids in the former is always very much smaller than that of the latter, provided both were deposited by identical geological processes. Sand-mica mixtures with macroscopic grains and with a high percentage of mica have not yet been found in Nature. The so-called "micaceous sands" as a rule contain only a very small fraction of mica by weight. The micaceous appearance is due to the fact that a single sand grain weighs twenty or more times that of a mica flake of equal size.

Mr. White is inclined to doubt the validity of the writer's statements concerning the effect of the depth of a foundation on the settlement for perfectly cohesionless material. His doubts are unquestionably based on what he has observed in actual practice where he has had almost no chance to study a material conforming with the condition of perfect lack of cohesion.

Mr. White's examples of the general character of the settlement of whole buildings (Fig. 31) leave little doubt that the centers of the outside walls settled more than the corners, but it is still an open question whether the centers of these buildings settled more than their perimeters. In order to insure uniform settlements, some engineers in Texas follow the practice of keeping the soil pressures for the outer footings 12 to 20% less than those for the inner footings, and their experience seems to confirm the soundness of their practice. It should also be considered that the relation between the amount of settlement of the center and of the perimeter of a building, at equal

* Abstract by Charles Terzaghi, M. Am. Soc. C. E., in "Economic Geology," 1928.

soil pressure per unit of area may be very different, depending on the degree to which the cohesion of the underground dominates over internal friction.

The introductory remarks of Mr. Simham's discussion describe exactly what the writer has been "driving at" for many years. The writer also concedes that no laws can be derived from uncorrelated sets of experiments; but, at the same time, he seriously doubts whether any formula can be derived to cover all the requirements of actual practice. In dealing with foundation problems, the writer confines himself to what Mr. Simham calls "a careful diagnosis of every factor that influences the behavior of a soil", and in doing so he has almost invariably found that he must be satisfied with finding out to what extent his previous experience applies to new cases. All the rules which he derived, and very likely all those which he will derive in the future, are merely intended to be tools for use in this fundamentally important diagnosis.

Equations (12) to (21), inclusive, represent one possible approach among many to a mathematical description of what engineers may observe. However, their value will depend on the degree to which Mr. Simham succeeds in taking care of the time factor, which is the stumbling block in the path of all similar attempts.

Mr. Marston describes an example for the application of soil physics to the study of materials for hydraulic-fill dams. Since this investigation was terminated, much progress has been made along similar lines by the work performed in the writer's laboratory. In pursuing this work, the thorough investigation of perfectly undisturbed materials extracted from the core of an existing hydraulic-fill dam proved to be of inestimable assistance and threw much light on the disputed problems. (See, also, the writer's comments on Dr. Chatley's discussion).

Mr. Shaw describes interesting experiences with New Orleans muck soils. Since the internal friction of under-drained muck soils is extremely low, their bearing capacity depends essentially on their cohesion. At the same time, that they are fairly permeable is proved by the fact that it is possible to drain them with open ditches. For soils of such a character the rule represented by Curves C and C_1, Fig. 50, has at least approximate validity. This statement agrees with Mr. Shaw's experiences.

Furthermore, previously published theoretical investigations of the writer have led to the following conclusions regarding the ultimate bearing capacity of medium permeable and saturated silts and muck soils. Select on the surface of such a soil two equal areas and load both of them at a different time rate until failure occurs by an "out-and-up" flow of the loaded soil. The more slowly the load is applied, the more chance the soil has to expel its excess water during load application. Therefore, ultimate failure of the area which was loaded very slowly will occur at a higher load per unit of area than that on which the load was rapidly applied. This theoretical conclusion is substantiated by the success of Mr. Shaw's method of levee construction by successive application of wide and thin layers. The compacting effect produced by load application is illustrated by the writer's Fig. 4.

Mr. Shaw's observations concerning the difference between the effect on soft soils of concentrated and of distributed loads call attention to an impor-

tant fact, closely related to the influence of the depth of foundation on the ultimate bearing capacity of the soil. Suppose the area, *a-a* (Fig. 51), is uniformly loaded, while on both sides of *a-a*, the load decreases from its value at *a* toward zero. For this case, both theory and experience lead to the conclusion that the wider the area over which the load spreads, the greater is the load, p, p_1, p_2, etc., at which failure occurs.

The effect of the flaring out of the load becomes still more conspicuous if the thickness of the soft layer is very much less than the width of the base of the fill (Fig. 52). In this case, the soft layer is "trapped" between the base of the load and the surface of the stiff layer. If the fill has no tensile strength, the limited depth of the soft layer and the friction along the top surface of the firmer ground introduce factors of resistance absent in deposits of greater depth; but if the fill has tensile strength, an arching effect, similar to that which takes place in grain bins, still further reduces the tendency of the soft material to flow. (See Fig. 52.) From his experiences with the construction of fills on peat deposits in Michigan, Mr. V. R. Burton* concluded that in placing a fill with a width ranging between 24 and 30 ft. on the surface of a peat deposit, the character of the resulting subsidence was very different, for varying thicknesses of the deposit. If the thickness was less than about 20 ft., the fill merely produced a trough-like depression in the peat deposit. On the other hand, if the thickness of the deposit exceeded about 26 ft., the fills penetrated as far as the bottom of the deposit, thus completely displacing the peat beneath the center line of the fill. Examples of the sudden subsidence of fills placed on soft muck soils can be found in the report of the Swedish Geo-Technical Commission.† The observations made by the Swedish engineers in connection with large-scale subsidence failures led them to base their stability computations for such soils on the assumption of circular sliding curves.

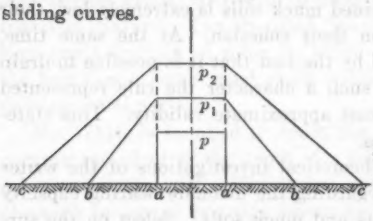


FIG. 51.—INCREASE IN BEARING CAPACITY OF UNDERGROUND DUE TO WIDENING FLARE OF TRAPEZOIDAL LOAD.

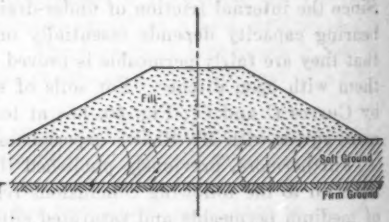


FIG. 52.—ARCHING EFFECT WITHIN A THIN, SOFT LAYER LOCATED BETWEEN BOTTOM OF FILL AND TOP SURFACE OF FIRM GROUND.

Mr. Shaw's experiences furnish instructive examples of the fact that the results obtained by theory can be depended upon, provided the assumptions conform reasonably well with the conditions existing in Nature.

It was a great pleasure to survey the valuable test results presented by Mr. Wing. In regard to the loading tests the writer would consider the tests

* "Fill Settlements in Peat Marshes," by V. R. Burton, *Proceedings*, Sixth Annual Meeting, Highway Research Board, 1926, pp. 93–113.

† Statens Jarnvägars Geotekniska Comm., 1914–22. An English abstract and comments on the report by Charles Terzaghi, M. Am. Soc. C. E., are filed in the Carnegie Library, Pittsburgh, Pa.

finished as soon as the settlement becomes equal to about one-fifth the diameter of the loaded area. Beyond this limit the process of settlement passes into one of penetration. When making the tests oil-jacks are always preferable to screw-jacks.

The writer uses as a criterion for the bearing quality of sand deposits the settlement under a load of 500 lb. per sq. ft., provided the loaded area is at least equal to 1 sq. ft. Under such a unit load, a well-compacted sand deposit does not settle more than about 0.001 ft. The loosest deposit that ever came under the writer's observation settled under the same load through a distance of $\frac{1}{4}$ in. From the character of the settlement curves for the sand (Fig. 44), the writer suspects that the sand was loosened by the upward current of water, as the pit was being pumped dry. On account of this possibility, the writer keeps the water out of a test pit by pumping, uncovers two opposite vertical sides of the pit, and makes the loading test in a horizontal direction, by inserting a hydraulic jack between two bearing plates, applied to the sides of the pit. The bearing plates should have a clearance, all around the rim, equal to at least twice the diameter of the plate. Experience has shown that the results of such tests differ but little from those made on horizontal plates, and one is sure that the loaded material is in an undisturbed state.

The results of mechanical analysis should preferably be presented in graphical form with the grain size plotted on a logarithmic scale.* The limits of consistency of the clay, quoted by Mr. Wing, remove any doubt concerning the type of clay that he encountered. Experience has shown that, for natural clay deposits, both the limits of consistency and the water content vary within rather wide limits. Therefore, it is advisable to perform these tests for each drill hole on several samples. Instead of determining the volume change associated with a change of the moisture content, it is preferable to determine the shrinkage limit (from dry weight and from the volume of the dried sample, assuming for the dried substance a specific gravity of 2.7).†

The settlement diagram (Fig. 45), for the clay indicates that the loaded area was too big for the size of the test pit. The abnormal shape of the curve seems essentially to be due to arching action within the clay, between the bearing plate and the walls of the pit. Similar phenomena were observed by the writer on a small scale, when loading clay specimens that were contained in a rigid box. The clay beneath the bearing plate undoubtedly compacted, but this fact alone does not account for the characteristics of the curve. The width of the pit should be equal at least to five times the diameter of the loaded area.

Loading tests on both sands and clays should always be made at least in duplicate on what appears to be the stiffest and the softest spot of the underground. The writer selects these spots after having passed over the ground with a sounding rod. The observations to be performed on the diversion dam are most promising, particularly if the settlement observations extend over a longer period and if, in addition, bench-marks are established on the unoccupied ground at a distance equal to one-half the width of the structure.

* "Physical Differences Between Sand and Clay," by Charles Terzaghi, M. Am. Soc. C. E., *Engineering News-Record*, December 3, 1925, Fig. 2.

† "Present Status of Subgrade Testing," by C. A. Hogentogler, Charles Terzaghi, and A. M. Wintermayr, *Public Roads*, March, 1928.

In his very thorough and able discussion, Professor Krynine quotes certain statements made by Dr. Ing. Koegler concerning the relation between settlement and size of area, and finds them to be at variance with those of the writer, as far as soils with great cohesion are concerned. It should be emphasized that Dr. Koegler derived his conclusions exclusively from tests performed with cohesionless sand. For this reason they conform to Curves C and C_2, Fig. 50. For the same reason they certainly do not apply to soils in general, irrespective of their physical characteristics, and Dr. Koegler's generalized statements must be received with reserve until more convincing arguments are presented for substantiating their validity.

Professor Krynine's suggestion to consider, in the computation of a raft foundation, the presence in the ground of hard and soft spots is theoretically sound. However, in practice, it seems very difficult to follow the advice. First, loading tests would be required, not only all over the area, but also at different elevations below the ground surface. Furthermore, since engineers do not yet know the distribution of the soil reactions for perfectly homogeneous foundations, and the importance of the effect of the soil characteristics on this pressure distribution, there is little hope for successfully introducing further refinements into computations.

The doubts expressed regarding the writer's comment on the settlement of the hardpan caissons at the Chicago Union Terminal, were apparently due to the fact that Professor Krynine was not familiar with the details of the tests. The frictional resistance acting over the surface of the caissons was determined by undermining the caissons and loading them until they slipped. The unit loads quoted by the writer obviously represented the top load minus the relieving effect of side friction.

Another objection by Professor Krynine concerns the shape of the curve which represents the distribution of the soil reactions over the base of a loaded slab. When the writer stated that these curves have a parabolic shape, he obviously meant an approximately parabolic one. At the rudimentary state of present knowledge a more specific statement concerning this shape would be premature. In addition, the difference between the right and left sides of Fig. 40 is so slight compared with the importance of the errors associated with Professor Krynine's theoretical assumptions that it seems reasonable to disregard it. Very much more important is the question as to whether the parabolic law may not lose its validity altogether if applied to very cohesive soils. For such soils the pressure distribution is apt to be similar to that over the base of a piston which acts on the plane surface of a homogeneous, elastic solid. The writer doubts seriously whether this distribution has any resemblance at all with either one of the distributions represented by Fig. 40.

Professor Krynine's comments on Fig. 5 are fully justified. The writer agrees that the two lines will be neither straight nor parallel. The reason he failed to show their true shape in the diagram was simple enough. He does not yet know this shape.

The writer became interested in the subject of "entrapped air" for the first time when certain inconsistencies were found to exist between the results of compression tests on clays that were dried before testing and on others that

were used as they were taken from the drill hole. The inconsistencies were caused by the fact that those samples which were prepared by mixing the dried specimen with water contained an appreciable quantity of air, while the others contained practically none. In order to determine the importance of the influence of the air on the test results, Mr. A. Casagrande performed some tests in the writer's laboratory. Fig. 53 shows a typical test result. One of the two samples merely contained air and water. The other was prepared from dry powder and, in addition, air was injected into the specimen by a drop counter in such quantities that its voids ratio increased by almost 50 per cent.

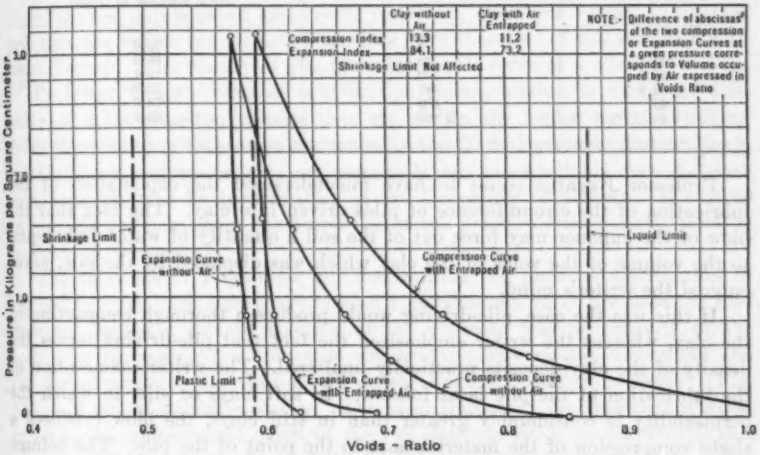


FIG. 53.—EFFECT OF AIR CONTENT ON ELASTIC PROPERTIES OF CLAY.

The most conspicuous effect of the air content was the difference in shape of the time-compression curves of the samples. This is caused by the fact that the compression of the air takes place almost instantaneously, similar to the compression under pressure of the air bubble in a spirit-level, while the compression of the clay occurs gradually, according to the laws of consolidation of compressible capillary systems. By this very reliable criterion, and by the results of numerous air-content determinations, the writer has learned that neither clays nor silts seem to contain in Nature a measurable amount of air, provided they are beneath the zone of temporary desiccation. For this reason the writer did not consider it necessary to mention the air factor in connection with a paper on foundations. It plays an important part merely in connection with sub-grade studies for roads. The intermittent wetting of the subgrade of hard surface roads occurs almost entirely by capillary action, whereby a considerable quantity of air remains in the soil.

Some experiments on the air content of fine sands and dust, wetted by capillary action, were conducted in 1926 by Mr. C. V. Arellano, under the supervision of the writer.* The test results seemed to indicate that the air

* "Relation between Permeability of Sands and Capillary Upward Movement of Water," by C. V. Arellano, Master's Thesis, Mass. Inst. Tech., 1926.

content decreases considerably with time. One of the samples tested was a fine, reddish, micaceous dust; effective size, 0.028; uniformity coefficient, 3.6; specific gravity of grains, 2.674; voids ratio, 0.698; and coefficient of permeability, 9.75×10^{-5} cm. per sec. The results are shown in Table 12.

TABLE 12.—AVERAGE DEGREE OF SATURATION OF FINE DUST WETTED BY CAPILLARY RISE.

Capillary rise, in centimeters.	Time elapsed since water started to rise, in minutes.	Percentage saturation, or the ratio between total volume of water and total volume of voids.
5	9	75.2
10	38	69.9
15.7	100	70.1
32.9	487	72.2
40	800	73.0
40	3 800	82.7

Professor Krynine seems to have misunderstood the explanation of the lubrication of the circumference of piles driven into clay. The idea that the blow of the hammer may force out of the soil a quantity of water fully equal to the volume of the voids of the clay which was displaced by the pile, never entered the writer's mind.

If this was the case, pile-driving would produce a thorough compacting of the clay, whereas the writer emphasized the fact that pile-driving leaves the density of the clay deposits practically unaltered. The writer's conception of the lubrication of the pile is as follows: In soft clays or silts in which the permeability is considerably greater than in stiff clays, the blow produces a slight compression of the material next to the point of the pile. The volume of water pressed out of the material represents a small fraction only of the volume displaced. Lubrication is caused by this small quantity escaping between the soil and the surface of the pile. The absorption of the film during the period between driving and re-driving is due to subsequent expansion of the slightly compressed clay which surrounds the pile. This process of absorption by no means requires (as Professor Krynine assumes) the presence of entrapped air, because it also occurs in the laboratory under test conditions which completely exclude the presence of air in the clay.

There is also another factor that is apt to cause a gradual increase of the "skin friction". If certain clays and organic muds are moulded into cylindrical shapes and tested at different times after moulding, one is apt to find that the shearing strength increases considerably as the time interval becomes greater. This increase seems to be due to the gradual development of a molecular bond between the individual particles across the layers of absorbed material. If piles are driven into such materials, the skin friction should also increase as time goes on.

Professor Krynine emphasizes the leaning of Russian engineers toward the "dry substance theory". On account of this leaning, Russian engineers seem to be inclined to speak of the "bearing capacity of soils" in general without due regard to their extremely manifold physical characteristics. This, at

least, is the impression the writer has obtained from some recent Russian publications on this subject.* More than a half century ago, a prominent French engineer expressed his experiences with earth slides in a single sentence which may be translated: "The fear of frost and of clay marks the beginning of wisdom". In a similar fashion the writer is inclined to summarize his experiences in dealing with the bearing-capacity problem in the sentence: "To recognize the full importance of the permeability factor marks the beginning of wisdom in foundation engineering". Since the permeability factor represents by far the most obstinate stumbling block in the path of any attempt to bridge the gap between small-scale tests with clays and actual foundation practice, the writer cannot help regarding "dry theories" of soil behavior with deep distrust. The history of pile formulas should serve as a solemn warning.

Professor Enger's valuable contribution calls attention to the great difficulty of measuring the stresses that act within the loaded earth. Judging from the results of measurements made by the U. S. Bureau of Public Roads the Goldbeck pressure cells do not seem to be any more reliable than the pressure capsules used by investigators at the University of Illinois. It will be interesting to learn about the properties of the pressure cells recently used by Koegler in Germany.†

The formulas of Boussinesq for determining the pressure distribution in the soil beneath loaded areas are at least approximately valid, provided the load does not exceed a certain fraction of the ultimate bearing capacity of the soil. For expressing the difference which exists between the theoretical and the actual pressure distribution, Professor Enger's "lines of pressure" (Fig. 48), will render valuable service. The most important problem that remains to be solved concerns the effect of the soil characteristics on the distribution of the pressures over the base of the loaded area.

The objection of Professor Enger against the use of remixed material does not seem to be quite justified. In the example which he cites, the compacting of the clay was performed by the capillary pressure at a time when the water-table was at a lower elevation. Once compacted, the clay takes up only a small fraction of the quantity of water which it previously lost. The process can be reproduced very easily in the laboratory by first compacting the clay under external or capillary pressure (applying the pressure under water or exposing the surface of the clay to the air) and then allowing it to expand very slowly under water.

In Nature one even encounters layers of very stiff clay beneath the bottom of lakes, between layers consisting of the same material in a very soft state. The history of these "fossil crusts" is as follows:‡ At a period when the surface of the older clay deposit was exposed to the air, the top layer of this deposit partly dried out (compression due to capillary pressure). At a later date, the surface of the older deposit was flooded, and a new soft clay deposit was formed on top of it. Due to the very imperfect volume elasticity of the

* Brief abstract of chapter on "Bearing Capacity," by Mr. N. Ivanoff, in Textbook on Soils and Top Soils, of the Russian Highway Research Bureau, Leningrad, 1926.

† "Ueber Tragfähigkeit von Sandschüttungen," *Bautechnik*, 1927.

‡ Statens Jarnvägars Geotekniska Comm., 1914–22, English abstract and comments by Charles Terzaghi, M. Am. Soc. C. E., filed in Carnegie Library, Pittsburgh, Pa.

clay, the former crust retained the greater part of its stiff consistency (see, for instance, the re-saturation or swelling curves in Fig. 53).

Since the paper was published, much headway has been made in developing methods for securing undisturbed samples of clay.* Tests are being planned to investigate the relative importance of the disturbance caused by different sampling tools. There seems also to be no difficulty in securing undisturbed clay samples from depths as great as 100 ft.

The effect of vibrations on the settlement of structures resting on clay was brought to the writer's attention under rather peculiar circumstances. Loading tests were made on a small scale on glacial clay, in his laboratory, and very sensitive instruments were used for measuring the settlements. All the settlement curves showed distinct breaks at points corresponding to 5:00 P. M. and 9:00 A. M., the 9:00 A. M. to 5:00 P. M. sections being considerably steeper than the 5:00 P. M. to 9:00 A. M. sections. The cause of this phenomenon was found to be that between 9:00 A. M. and 5:00 P. M. the different motors located in the Massachusetts Institute of Technology Laboratories are operating, while at 5:00 P. M., they are shut down.

The tests suggested by Mr. Jorgensen will be particularly valuable if they are made in triplicate, using identical bearing plates, resting on identical clay water mixtures, so that one of the bearing plates consists of cast iron (impermeable), one of concrete (medium permeable), and one of concrete separated from the clay by a sheet of filter paper and a thin layer of sand (permeable). The diameter of the bearing plates should not be more than one-fifth that of the container. The exposed surface of the clay should be covered with a paraffined rubber sheet in order to prevent evaporation, and the settlements should be measured with delicate instruments to detect the difference in the speed of settlement of the three plates. During the tests, the load should be kept constant, and should be equal to the pressure required to produce an initial settlement of the cast-iron bearing plate equal to about 0.05 or 0.1 times its diameter. In each of the three cases the load should be applied in equal increments and in equal time intervals. In order to obtain the maximum benefit from the tests, the base of the bearing plates should be equipped with annular oil-pressure cells, for investigating the distribution of the soil reaction over the case.

The procedure proposed by Mr. Jorgensen for load-moisture tests is practically identical with that of the writer. Since the shape of the re-compression curves, *c d* (Fig. 49), is only slightly influenced by the intensity of the pressure at which swelling was begun, no great care is necessary in performing the test.

In conclusion, the writer wishes to express his gratitude to the discussors for their valuable contributions. The gaps in the presentation of the subject are still numerous. Yet, the arguments brought forth offered a unique opportunity for expanding the paper on those points which were still in doubt in the minds of the readers.

* "Relation Between Permeability of Sands and Capillary Upward Movement of Water," by C. V. Arellano, Master's Thesis, Mass. Inst. Tech., Cambridge, Mass., 1926.

The results of the discussion may be summarized, as follows:

1.—The *Engineering News Formula* received all the comment it deserves and few engineers seem to be willing to defend it.

2.—The attitude of the different engineers toward the writer's theoretical statements represented by Fig. 50 was far less unanimous, yet it leads to very interesting conclusions because no obvious contradiction between theory and practice was found to exist by those who reported experiences with fairly permeable materials (Messrs. White, with friable limestone; Shaw, with muck; and Wing, with sand and gravel). Without exception, all the examples which were quoted as arguments against the validity of the writer's "rules" referred to clays. The reason thereof is obvious. The compression of clay under load proceeds very slowly. Large footings or raft foundations require decades or even hundreds of years for complete settlement, while the observations on which the arguments were based covered, at the very best, a period of a couple of years of the settlement records. In other words, the bill for the excessive loading of a sand or silt foundation should be paid without delay by the first generation, while that for treating the clay in a similar manner should be charged against the second or third generation.

It is amazing to realize that the Engineering Profession is facing the problem of progressive subsidence of clay foundations in almost every part of the globe, and yet is still completely ignoring the time element when it comes to approaching the settlement problem from the theoretical point of view. If the discussion had not brought forth anything but striking illustrations of this one fact, the labor involved could be considered as well invested.

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Paper No. 1705

MAXIMUM FLOOD DISCHARGE IN SAN JOAQUIN VALLEY, CALIFORNIA*

By Oren Reed,† Jun. Am. Soc. C. E.‡

With Discussion by Messrs. Robert Follansbee, Harold K. Fox, Charles H. Lee, and Oren Reed.

Synopsis

Much has been written on the subject of floods in the last few years and many formulas have been presented for general or local application. Since the effect of similar storms on a given water-shed may vary widely, depending on temperature and storage conditions, formulas must be used with caution. In this paper an attempt has been made to estimate flood discharge for one river from the known discharges of rivers of the same meteorological province.

In each meteorological province there seems to be a type storm, the characteristics of which are relatively constant from time to time. Records of several streams in such a province, similar in storage, vegetation, soil, and other physical characteristics, can be combined, after the proper adjustments are made, to give an equivalent record for the stream under consideration.

When long-term records are available each drainage area may be considered by itself, but records of many years are required to estimate the extreme flood to be expected on a stream. The method may be used to estimate flood discharge of a stream, where no flow measurements have been made, by comparison with the known discharges of neighboring water-sheds.

Flood Characteristics

In many localities stream-flow records are few or entirely lacking; then the engineer must rely on a study of the evidences of former floods, and

* Published in November, 1927, *Proceedings*.
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on the conflicting stories of inhabitants of the neighborhood. As an aid in flood study many different formulas have been proposed for computing probable flood run-off. Some are only applicable to the small area for which they were originally devised on account of local conditions of soil, soil cover, slope of drainage, storage, etc. The use of most flood formulas is dangerous unless due regard is given to their basic limitations.

Except for small areas, the great floods in the Pacific Coast section occur in the winter or early spring and are caused by heavy rainfall. On the larger streams, the drainage areas of which extend to high altitudes, the flood stage may be augmented, in some years, by the melting of snow.

In winter, an area of low barometric pressure exists over the North Pacific Ocean. As the great cyclonic storms sweep easterly or southeasterly from the Pacific, they yield gentle but persistent rainfall. After the storm center has passed and the wind swings from southeast to southwest, the rainfall increases in intensity for a short time; the winds then swing to the northwest and the storm has passed. In the early winter and late spring the storms follow each other in rapid succession, and heavy precipitation from these successive storms tends to cause extreme flood conditions. In the early part of winter, when a storm period that produces a considerable depth of snow is followed by a series of warm rains, a high flood is to be expected. If the snow becomes packed its water content is greatly increased, but high run-off will only result after several days of warm weather. With the rising temperature of spring, the dense snow cover starts to melt, augmented by short storms. This spring run-off seldom causes as high peaks as the mid-winter flood period, but gives a long-sustained stage of more moderate proportion.

Exceptions to these general rules are the spring-fed streams flowing from the lava country of Northern California, Eastern Oregon, and Southern Idaho, and small cloudburst areas, as in Oregon. Streams from the lava country are characterized by run-off of a marvelous constancy.

Flood Variation

While making an investigation to determine the probable maximum flood of the North Fork of Kings River, for the San Joaquin Light and Power Corporation, at Fresno, Calif., the writer was confronted with a maze of factors influencing the problem. The results of several formulas could not be reconciled, and it was decided to study previous floods on the Kings River at Sanger and on ten other major streams draining the west slope of the Sierra Nevada Mountains (Fig. 1), namely, the Cosumnes, Mokelumne, Stanislaus, Tuolumne, Merced, Fresno, San Joaquin, Kaweah, Tule, and Kern Rivers. Proper factors were determined to apply to a flood on one stream to make it applicable to Kings River.

To be strictly applicable to a particular drainage area, a formula for flood run-off must be modified to suit local conditions. General rainfall and run-off characteristics are relatively constant in each type province, but minor differences will be numerous. Small tributary streams may give widely different flood peaks on account of variable topographic conditions. Rancheria Creek, a tributary of the North Fork of Kings River, having a drainage area

of about 25 sq. miles, is an example of low flood variation. The average elevation of the area is about 8 000, and the total run-off per square mile is high. However, the stream is not flashy. It drains a northwest slope, and the forest cover of pine and spruce is heavy. Until the stream reaches the canyon of the North Fork of Kings River, the slope is moderate and the drainage is retarded by many lakes and meadows.

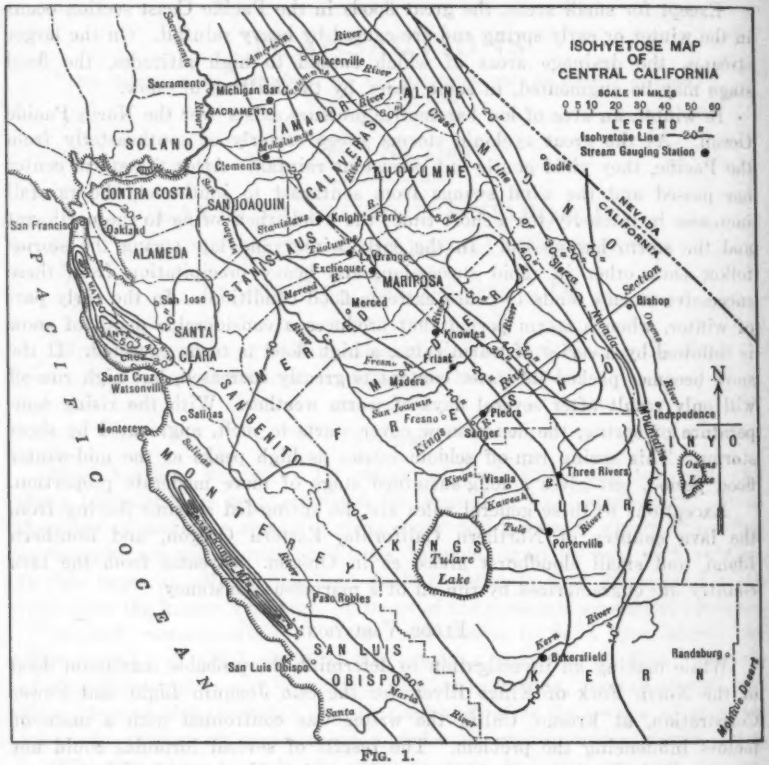

FIG. 1.

PHYSICAL FEATURES

General.—The streams chosen for this investigation are tributary to the San Joaquin River or discharge into the Tulare and Kern Lake Basins and, together, drain the west slope of the Sierra Nevada Mountains and discharge into the southern half of the Great Valley of California. The Central Valley is a deep structural trough filled with alluvial material derived principally from the débris brought down by those streams from the Sierra Nevadas. The Sierras are a granitic formation with small areas of volcanic material. The streams rise at or near the divide and flow by a rapid gradient and in deep canyons to the outwash plain. The rainfall decreases toward the south, but the precipitation characteristics are similar on all eleven streams considered (Fig. 1).

Kern River.—Kern River, the most southern of the great rivers that flow westward from the crest of the Sierras, has a long, narrow basin. The general trend of the basin is north and south, except at the lower end. At the head-waters is the Mt. Whitney region, the highest and roughest in the Sierras. The main or North Fork of the Kern River is very rugged, whereas the South Fork is more flat in its lower basin. In general, the forest cover is very poor.

Tule River.—The Tule River heads on the Great Western Divide, the secondary ridge west of the Kern River. Part of the upper basin is well forested, but a large part of the entire basin is densely covered with chaparral and scrub oak.

Kaweah River.—The head-water region of the Kaweah is very rugged, but does not reach to the main divide. The ridges of the lower country, especially on the Marble Fork, are well forested. The flow of short rivers, like the Kaweah and the Tule, is decidedly flashy, owing to the extremely steep gradients, the relatively small drainage areas, and the predominant granite formations.

Kings River.—The Kings River rises at the crest of the Sierras near the Mt. Whitney group. The South and Middle Forks have steep gradients near their sources and have carved out profound canyons. Their slopes in the lower courses are more moderate. In general, the forest cover is good, although it has been cut over in certain areas.

The head-waters of the North Fork drain the west side of Le Conte Divide, Mt. Reinstein (12 575 ft.) being the highest elevation. The upper part of the basin is an open plateau and the slope is moderate, but in the lower course the stream drops at a tremendous gradient to the main river. The lower basin is covered with scrub timber; above Elevation 4 000, there is an excellent stand of timber which extends to the timber-line near the head-waters.

San Joaquin River.—The mountain basin of the San Joaquin has the form of a rough isosceles triangle with a base fifty miles long, extending northwest and southeast along the crest of the Sierra Nevada Mountains. The course of the main stream is, in general, southwest, while the principal tributaries near its head-waters flow through troughs trending parallel to the main divide. The brush-covered foothills give place to well-forested ridges, extending up about 9 500 ft., above which the cover thins out and the upper basin is entirely bare. Some storage has been developed in the upper part of the basin: Crane Valley Reservoir, 45 000 acre-ft.; Huntington Lake, 88 000 acre-ft.; and Florence Lake, 60 000 acre-ft.

Fresno River.—The Fresno River does not reach the main divide, but heads on the secondary ridges west of the San Joaquin River.

Merced River.—The Merced River rises at an elevation of about 11 000 ft. on the west slope of the secondary ridges, west of the head-waters of the Tuolumne and San Joaquin Rivers, its basin touching the main divide of the Sierras at only one point, Mt. Lyell (13 090 ft.). The topography of the head-water region is extremely rugged, but below Yosemite Valley it becomes

more regular and the stream is shut in by two high brush-covered ridges. Much of the middle basin, especially on the South Fork, is well forested. Exchequer Reservoir, constructed near the foothills, has a capacity of 280 000 acre-ft.

Tuolumne River.—The topography of the head-waters of the Tuolumne is very rugged. The Hetch Hetchy Valley, which is in the upper basin, rivals the Yosemite Valley in grandeur. Below Hetch Hetchy Valley the water-shed is more open. Most of the drainage area is well covered with forest interspersed with bare granite ridges and mountain meadows. Two reservoirs have recently been completed on the Tuolumne—Don Pedro, near the foothills, 270 000 acre-ft., completed in 1923, and O'Shaughnessy Reservoir, 206 000 acre-ft., at the lower end of Hetch Hetchy Valley.

Stanislaus River.—The Upper Stanislaus Basin is typically Alpine, including tremendous domes and peaks of granite and very rugged topography. In the lower course the stream is shut in by two broad-topped ridges which are covered with heavy timber. Near the head-waters of the basin there are several small storage reservoirs; Relief Reservoir, 14 965 acre-ft., and Strawberry Reservoir, 17 900 acre-ft., are the largest.

Mokelumne River.—The Mokelumne Basin is long and comparatively narrow, and extends nearly due west from the crest of the Sierras to the eastern edge of the foothills. Round Top (10 430 ft.) is the highest peak. Except in the higher regions, the country presents a spacious and rolling, rather than rugged, appearance. The basin is well forested, except near the head-waters which is for the most part of bare granite.

Cosumnes River.—Like the Mokelumne River, the Cosumnes Basin is long and narrow; for a great part of their length the tributaries flow in nearly parallel canyons. The highest peak is Alder Hill (7 875 ft.).

Precipitation and Seasonal Run-Off

As an aid in interpreting flood data, a study was made of seasonal precipitation and run-off. An isohyetose map of the south-central part of the State was drawn (Fig. 1).* Records of mean precipitation of 114 stations were used. The number of years included in the record at each station varied from fifty-four at Stockton and Santa Barbara to only four for the stations established by the San Joaquin Light and Power Corporation on the North Fork of Kings River. Four, or even ten, years are too few to determine true averages, but some short-term records were used to show the general trend. The records of the stations on the North Fork of Kings River were adjusted somewhat by a comparison with the records of the corresponding years at Huntington Lake on the adjoining San Joaquin water-shed.

Except for thunder-storms, which occur in the mountains in the late summer and early fall, precipitation on the west slope of the Sierras results

* The base for this map is the General Land Office Map of California. Isohyetose lines are based on precipitation as given for various stations in *Bulletin No. 5*, California Department of Public Works, except those for the San Francisco Bay region, which are as given by C. E. Grunsky, Past-President, Am. Soc. C. E., in *Transactions*, Am. Soc. C. E., Vol. LXXX (1916), p. 104.

from the movement of great cyclonic storms. The general track of these storms is from northwest to southeast. They enter the State from the Pacific Ocean near the northwest corner and, after crossing the Coast Range, follow a path roughly parallel to the trend of the Sierras. Drainage areas near the north end of the region considered—the Stanislaus and the Mokelumne—receive heavy precipitation. As the storm passes southward the atmosphere becomes dryer and beyond the Kings River Basin the decrease in precipitation is rapid. The southern water-sheds are rarely visited by cyclonic storms from the southwest. The variation in precipitation from north to south is shown by Fig. 2, which is based on three groups of stations, each group being in a definite elevation belt. During a wet winter, as in 1916, the storm center is shifted southward and the Kern and Tule Rivers receive much heavier precipitation.

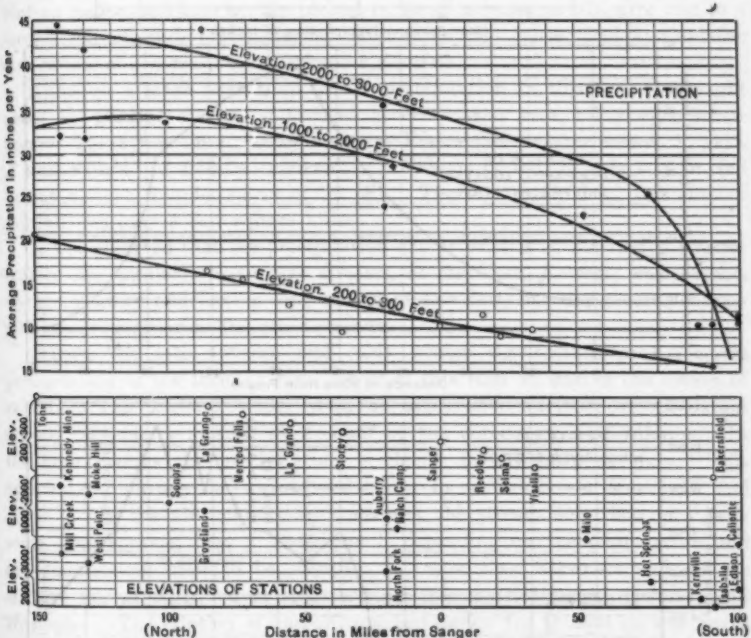

FIG. 2.—DISTRIBUTION OF PRECIPITATION FOR STATIONS AT VARIOUS ELEVATIONS IN SIERRA NEVADA MOUNTAINS, CALIFORNIA.

When precipitation records on a section at right angles to the trend of the Sierras are studied, it is seen that, to a certain elevation, precipitation increases with height. Beyond this, there is a slow decrease to the summit and a rapid decrease east of the mountains (Fig. 3). This Kings River Section is indicated on Fig. 1. The curves plotted for this section support the conclusions* of C. H. Lee, M. Am. Soc. C. E., that topography rather

* "The Determination of Safe Yield of Underground Reservoirs of the Closed-Basin Type." *Transactions*, Am. Soc. C. E., Vol. LXXVIII (1915), p. 148.

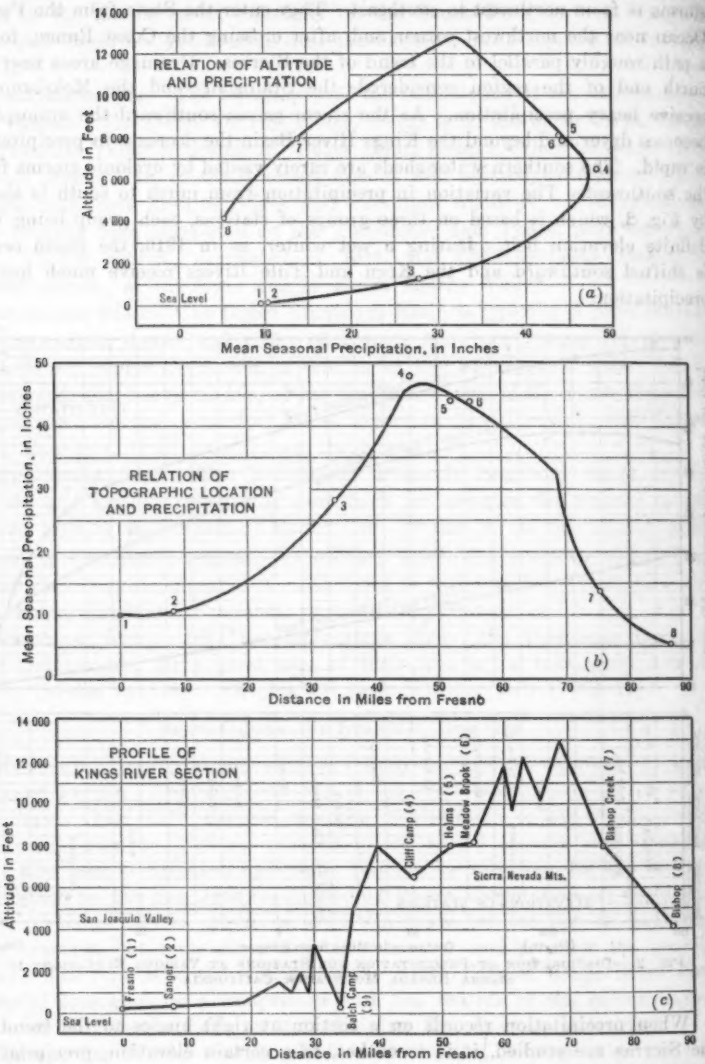

FIG. 3.—INFLUENCE OF ALTITUDE AND TOPOGRAPHIC LOCATION ON PRECIPITATION.

than elevation is the controlling factor in the distribution of total precipitation in the Sierra Nevada Mountains.

The greatest rainfall occurs at an elevation of 4 000 to 6 000 ft., differing with each storm, due to variation in relative humidity and temperature. The moisture-laden winds, which are blown against the mountain slope, are forced upward, giving rise to rapid cooling and precipitation. Heat is liberated with condensation and, as a result, the dew-point of the atmosphere is raised above this belt of maximum precipitation. The precipitation decreases gradually to the summit of the range, and as the winds pass over the divide, their absolute humidity having been decreased by the induced rainfall, they are compressed in their flow down the mountain with a further increase in temperature and, in consequence, become dry. If, after the winds pass through the lower area of maximum precipitation, they flow over a plateau region and lose heat, a second range of mountains will give rise to a second belt of high precipitation. Mountains also cause local atmospheric circulation and consequent orographic rainfall.

Although a large proportion of the precipitation in this area is brought about by general storms, sometimes severe storms resembling cloudbursts occur. There was an unusual storm at Fresno on October 4 and 5, 1925. A total rainfall of 1.56 in. occurred in a 36-hour period; of this 1.18 in. fell in one violent thunder-storm of 32 min. The area covered by this intense rainfall was only a few square miles.

It was observed that the precipitation characteristics of the twelve streams studied are similar. Secondary streams, as the Tule, Kaweah, and Fresno, are more flashy and have a long period of low flow, but the general characteristics are similar to the other streams.

During the late autumn and early winter the foothill area receives precipitation in the form of rain. Most of this runs off during the season of rainfall. The outwash slopes yield no appreciable surface run-off, except during very heavy storms, on account of the porous gravel formation. In the high Sierras the precipitation is chiefly in the form of snow. This accumulates during the winter and starts to melt between March 15 and April 1. As the air temperature increases, the stream flow rises to a peak some time between May 1 and June 1, depending on the temperature and the quantity of snow to be melted. The flow then decreases and all the streams are at their lowest stage during August, September, October, and November. The relation of run-off to precipitation for 1922-23 on the North Fork of Kings River above Balch Camp is shown in Fig. 4. This gives the true relation, although the distribution of precipitation is unusual.

When mean yearly run-off, in inches, on the drainage area was studied in relation to the location of the water-shed north or south of Sanger and to the average elevation of the drainage area, some interesting features of the drainage areas were observed. The "Distance from Sanger" curve (Fig. 5) was similar to Fig. 2, as was expected, but the low yield of the southern streams, the Kern and the Tule, was very apparent. Precipitation stations on many of the streams are in the foothills and their records are a

poor basis for determining the average precipitation on the drainage basins as a whole. The Fresno is a secondary stream with no tributary area of high elevation. The highest elevation of the water-shed is 6 350 ft. The relation of run-off to area of drainage (Fig. 6) showed more clearly the deficiency of run-off of the Kern, Tule, and Fresno.

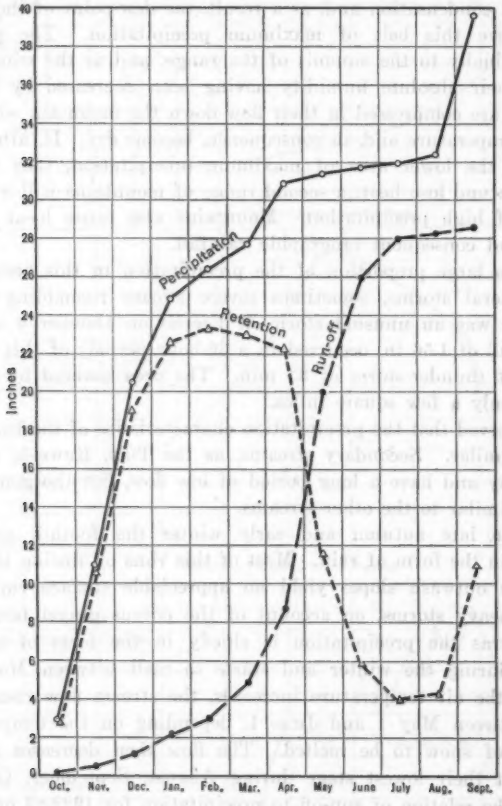

FIG. 4.—MASS DIAGRAMS OF RUN-OFF. PRECIPITATION RELATIONS ON NORTH FORK OF KINGS RIVER AT BALCH CAMP, 1922–23, SLIGHTLY BELOW NORMAL.

FLOOD DATA

The flood data used in this paper were taken from the *Water Supply Papers* of the U. S. Geological Survey. The primary data for the eleven California streams are given in Tables 1 and 2.

Gauging stations have been established by the U. S. Geological Survey on each of the rivers under consideration near the point where the stream leaves the foothills. The longest published record is for the Kings River near

Sanger, 26 years; the shortest record is for the Fresno, 8 years. The Kings River Gauging Station is at Piedra, a few miles northeast of Sanger. Until the seasonal year, 1913-14, only the maximum 24-hour flood was published.

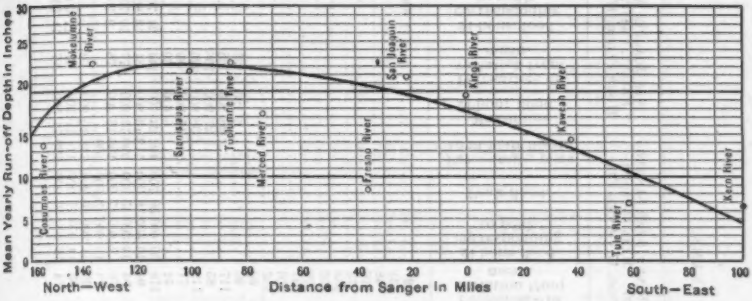

FIG. 5.—RELATION OF YEARLY RUN-OFF TO GEOGRAPHICAL LOCATION OF ELEVEN CALIFORNIA STREAMS.

In Table 1 the "24-hour flood corresponding to the peak flood" is given. In some years this peak flood did not occur on the same day as the maximum 24-hour flood, but came as a sudden and unsustained peak during the winter.

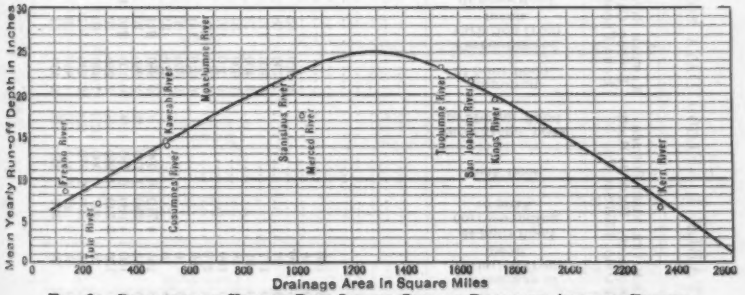

FIG. 6.—RELATION OF YEARLY RUN-OFF TO SIZE OF DRAINAGE AREA OF ELEVEN CALIFORNIA STREAMS.

FLOOD DETERMINATION

If sufficient data were available, the most accurate way of determining the maximum flood on any stream would be to consider each stream by itself. A record of at least 100 years on each stream would be necessary, in order to establish probable variations of flood run-off. Even that length of record might be inadequate, for maximum flows are often grouped close together with a long interval of medium, or low, flows. Flood records of the Danube River at Vienna, Austria, which cover 900 years, show that a record of 100 years is not sufficient to fix the maximum flood. Three extraordinary floods have occurred at Vienna with time intervals of 112 and 286 years. The greatest flood was one-third greater than the second record flood.

TABLE 1.—FLOOD DATA FOR CALIFORNIA STREAMS.

Seasonal year.	KERN RIVER, AT BAKERSFIELD, CALIF. MEAN 24-HOUR FLOOD, 5 093 SEC-FT.					TULE RIVER, NEAR PORTERVILLE, CALIF. MEAN 24-HOUR FLOOD, 2 556 SEC-FT.					KAWEAH RIVER, NEAR THREE RIVERS, CALIF. MEAN 24-HOUR FLOOD, 4 683 SEC-FT.					KINGS RIVER, NEAR SANGER, CALIF. MEAN 24-HOUR FLOOD, 17 358 SEC-FT.					SAN JOAQUIN RIVER, NEAR FRIANT, CALIF. MEAN 24-HOUR FLOOD, 17 118 SEC-FT.				
	Maximum 24-hour flood.	Percentage of variation from mean.	24-hour flood corresponding to peak.	Peak flood.	Percentage, peak to 24-hour flood.	Maximum 24-hour flood.	Percentage of variation from mean.	24-hour flood corresponding to peak.	Peak flood.	Percentage, peak to 24-hour flood.	Maximum 24-hour flood.	Percentage of variation from mean.	24-hour flood corresponding to peak.	Peak flood.	Percentage, peak to 24-hour flood.	Maximum 24-hour flood.	Percentage of variation from mean.	24-hour flood corresponding to peak.	Peak flood.	Percentage, peak to 24-hour flood.	Maximum 24-hour flood.	Percentage of variation from mean.	24-hour flood corresponding to peak.	Peak flood.	Percentage, peak to 24-hour flood.
1895–1896	.....	....	.....	.....	....	.....	....	.....	.....	....	.....	....	.....	.....	....	22 100	127	.....	.....	....	.....	....	(1 880)	60 000	....
1896– 97	5 342	103	.....	.....	....	.....	....	.....	.....	....	.....	....	.....	.....	....	22 732	131	.....	.....	....	.....	....	.....	.....	....
1897– 98	1 342	26	.....	.....	....	.....	....	.....	.....	....	.....	....	.....	.....	....	8 348	48	.....	.....	....	.....	....	.....	.....	....
1898– 99	4 932	97	.....	.....	....	.....	....	.....	.....	....	.....	....	.....	.....	....	20 200	116	.....	.....	....	.....	....	.....	.....	....
1899–1900	1 852	36	.....	.....	....	.....	....	.....	.....	....	.....	....	.....	.....	....	12 700	73	.....	.....	....	.....	....	.....	.....	....
1900– 01	4 295	85	.....	.....	....	.....	....	.....	.....	....	.....	....	.....	.....	....	43 930	253	.....	.....	....	.....	....	.....	.....	....
1901– 02	3 758	74	.....	.....	....	4 615	180	.....	.....	....	.....	....	.....	.....	....	26 380	152	.....	.....	....	.....	....	.....	.....	....
1902– 03	3 374	66	.....	.....	....	3 790	148	.....	.....	....	.....	....	.....	.....	....	17 290	99	.....	.....	....	.....	....	.....	.....	....
1903– 04	3 167	62	.....	.....	....	2 305	90	.....	.....	....	2 920	63	.....	.....	....	15 700	91	.....	.....	....	.....	....	.....	.....	....
1904– 05	3 039	60	.....	.....	....	822	32	.....	.....	....	2 730	58	.....	.....	....	9 795	56	.....	.....	....	.....	....	.....	.....	....
1905– 06	9 505	186	.....	.....	....	4 530	177	.....	.....	....	8 160	174	.....	.....	....	26 600	153	.....	.....	....	.....	....	.....	.....	....
1906– 07	.....	....	.....	.....	....	2 300	91	.....	.....	....	3 400	73	.....	.....	....	15 600	90	.....	.....	....	.....	....	.....	.....	....
1907– 08	.....	....	.....	.....	....	1 230	48	.....	.....	....	1 600	34	.....	.....	....	6 920	40	.....	.....	....	6 130	36	.....	.....	....
1908– 09	8 851	174	.....	.....	....	5 070	199	.....	.....	....	9 210	197	.....	.....	....	20 300	117	.....	.....	....	28 800	168	.....	.....	....
1909– 10	4 656	92	.....	.....	....	5 430	212	.....	.....	....	7 910	170	.....	.....	....	14 700	85	.....	.....	....	27 900	163	.....	.....	....
1910– 11	4 623	91	.....	.....	....	2 780	109	.....	.....	....	6 610	141	.....	.....	....	20 500	118	.....	.....	....	38 800	226	.....	.....	....
1911– 12	2 919	57	.....	.....	....	260	10	.....	.....	....	2 360	50	.....	.....	....	12 400	71	.....	.....	....	15 300	89	.....	.....	....
1912– 13	1 976	39	.....	.....	....	248	10	.....	.....	....	1 470	31	.....	.....	....	7 210	41	.....	.....	....	6 610	39	.....	.....	....
1913– 14	15 478	305	15 478	18 287	118	4 710	184	4 710	6 600	140	9 880	211	9 880	13 300	135	30 400	176	30 400	59 700	196	24 700	144	24 700	46 200	187
1914– 15	4 010	79	4 010	4 249	106	1 380	54	1 380	1 740	126	3 280	70	3 280	3 600	110	16 300	94	16 300	18 300	112	13 800	81	13 800	15 900	115
1915– 16	16 125	317	16 125	17 962	111	4 080	160	4 080	6 780	166	10 100	216	10 100	14 700	146	16 300	94	14 200	45 400	320	13 200	77	10 700	17 000	159
1916– 17	3 976	78	3 976	4 331	109	3 540	139	3 540	4 260	120	3 280	70	2 690	6 270	233	13 200	76	9 400	17 900	190	13 400	78	9 900	15 600	158
1917– 18	3 162	62	3 162	3 478	110	665	26	427	900	211	1 400	30	1 400	1 570	112	12 800	74	12 800	13 500	106	.....	....	.....	.....	....
1918– 19	3 851	76	3 851	4 299	111	665	26	500	1 070	214	2 690	57	2 690	2 880	105	11 200	65	11 200	13 200	118	10 900	64	.....	.....	....
1919– 20	3 657	72	3 449	4 456	129	2 140	84	2 140	3 450	161	4 850	103	4 850	5 850	121	14 900	86	14 900	17 000	114	11 500	67	11 500	14 000	122
1920– 21	3 247	64	3 247	3 548	109	565	22	565	657	116	2 450	52	2 450	3 190	130	12 800	74	12 400	16 000	129	11 500	67	11 500	15 300	133

TABLE 1.—(*Continued*).

Seasonal year.	Fresno River, near Knowles, Calif. Mean 24-hour flood, 1 807 sec-ft.					Merced River, near Merced Falls, Calif. After October, 1915, near Exchequer. Calif. Mean 24-hour flood, 13 178 sec-ft.					Tuolumne River, at La Grange, Calif. Mean 24-hour flood, 19 329 sec-ft.				
	Maximum 24-hour flood.	Percentage of variation from mean.	24-hour flood corresponding to peak.	Peak flood.	Percentage, peak to 24-hour flood.	Maximum 24-hour flood.	Percentage of variation from mean.	24-hour flood corresponding to peak.	Peak flood.	Percentage, peak to 24-hour flood.	Maximum 24-hour flood.	Percentage of variation from mean.	24-hour flood corresponding to peak.	Peak flood.	Percentage, peak to 24-hour flood.
1895–1896	.....	....	.....	.....	....	.....	....	.....	.....	....	11 798	61	.....	.....	....
1896– 97	.....	....	.....	.....	....	.....	....	.....	.....	....	14 700	76	.....	.....	....
1897– 98	.....	....	.....	.....	....	.....	....	.....	.....	....	7 800	41	.....	.....	....
1898– 99	.....	....	.....	.....	....	.....	....	.....	.....	....	21 800	113	.....	.....	....
1899–1900	.....	....	.....	.....	....	.....	....	.....	.....	....	13 160	68	.....	.....	....
1900– 01	.....	....	.....	.....	....	.....	....	.....	.....	....	19 240	100	.....	.....	....
1901– 02	.....	....	.....	.....	....	6 240	47	.....	.....	....	12 934	67	.....	.....	....
1902– 03	.....	....	.....	.....	....	11 400	87	.....	.....	....	20 342	105	.....	.....	....
1903– 04	.....	....	.....	.....	....	8 780	67	.....	.....	....	17 850	92	.....	.....	....
1904– 05	.....	....	.....	.....	....	8 020	61	.....	.....	....	15 855	82	.....	.....	....
1905– 06	.....	....	.....	.....	....	18 400	140	.....	.....	....	24 400	126	.....	.....	....
1906– 07	.....	....	.....	.....	....	27 500	209	.....	.....	....	52 000	270	.....	.....	....
1907– 08	.....	....	.....	.....	....	3 740	28	.....	.....	....	6 720	35	.....	.....	....
1908– 09	.....	....	.....	.....	....	19 500	148	.....	.....	....	26 700	138	.....	.....	....
1909– 10	.....	....	.....	.....	....	14 800	112	.....	.....	....	20 900	108	.....	.....	....
1910– 11	.....	....	.....	.....	....	37 200	282	.....	.....	....	52 600	273	52 600	60 300	115
1911– 12	189	10	.....	.....	....	6 100	46	.....	.....	....	13 800	72	.....	.....	....
1912– 13	118	7	.....	.....	....	3 180	24	.....	.....	....	6 320	33	.....	.....	....
1913– 14	.....	....	.....	.....	....	.....	....	.....	.....	....	31 200	161	.....	.....	....
1914– 15	.....	....	.....	.....	....	.....	....	.....	.....	....	13 300	69	13 300	17 200	129
1915– 16	3 770	208	2 570	4 010	156	12 600	96	7 240	22 000	304	15 400	80	15 400	20 500	133
1916– 17	3 770	208	3 770	4 500	119	18 500	140	17 400	21 700	125	22 900	119	22 900	36 500	159
1917– 18	2 490	138	2 490	2 710	109	13 200	100	10 500	17 700	169	15 100	79	15 100	23 300	154
1918– 19	1 740	96	1 740	1 940	111	7 740	59	7 740	8 540	110	13 500	70	13 500	15 600	116
1919– 20	942	52	530	970	183	7 360	56	6 320	8 950	142	12 900	68	11 000	15 500	141
1920– 21	1 440	80	1 440	1 740	121	13 000	99	13 000	19 400	149	.....	....	.....	.....	....

TABLE 1.—(*Continued*).

Seasonal year.	Stanislaus River, at Knight's Ferry, Calif. Mean 24-Hour Flood, 13 903 Sec-Ft.					Mokelumne River, near Clements, Calif. Mean 24-Hour Flood, 8 036 Sec-Ft.					Cosumnes River, near Michigan Bar, Calif. Mean 24-Hour Flood, 9 696 Sec-Ft.				
	Maximum 24-hour flood.	Percentage of variation from mean.	24-hour flood corresponding to peak.	Peak flood.	Percentage, peak to 24-hour flood.	Maximum 24-hour flood.	Percentage of variation from mean.	24-hour flood corresponding to peak.	Peak flood.	Percentage, peak to 24-hour flood.	Maximum 24-hour flood.	Percentage of variation from mean.	24-hour flood corresponding to peak.	Peak flood.	Percentage, peak to 24-hour flood.
1895–1896	.....	....	.....	.....	....	.....	....	.....	.....	....	.....	....	.....	.....	....
1896– 97	.....	....	.....	.....	....	.....	....	.....	.....	....	.....	....	.....	.....	....
1897– 98	.....	....	.....	.....	....	.....	....	.....	.....	....	.....	....	.....	.....	....
1898– 99	.....	....	.....	.....	....	.....	....	.....	.....	....	.....	....	.....	.....	....
1899–1900	.....	....	.....	.....	....	.....	....	.....	.....	....	.....	....	.....	.....	....
1900– 01	.....	....	.....	.....	....	.....	....	.....	.....	....	.....	....	.....	.....	....
1901– 02	.....	....	.....	.....	....	.....	....	.....	.....	....	.....	....	.....	.....	....
1902– 03	.....	....	.....	.....	....	.....	....	.....	.....	....	.....	....	.....	.....	....
1903– 04	.....	....	.....	.....	....	.....	....	.....	.....	....	.....	....	.....	.....	....
1904– 05	.....	....	.....	.....	....	5 260	65	.....	.....	....	.....	....	.....	.....	....
1905– 06	.....	....	.....	.....	....	8 740	108	.....	.....	....	.....	....	.....	.....	....
1906– 07	.....	....	.....	.....	....	15 300	191	.....	.....	....	.....	....	.....	.....	....
1907– 08	.....	....	.....	.....	....	2 960	37	.....	.....	....	2 180	23	.....	.....	....
1908– 09	36 600	263	.....	.....	....	10 400	130	.....	.....	....	20 800	216	.....	.....	....
1909– 10	9 880	71	.....	.....	....	7 200	90	.....	.....	....	7 200	74	.....	.....	....
1910– 11	36 900	266	36 900	60 200	163	16 700	208	16 700	20 500	123	22 400	232	.....	.....	....
1911– 12	5 880	42	.....	.....	....	4 920	61	.....	.....	....	1 100	11	.....	.....	....
1912– 13	4 380	31	.....	.....	....	3 840	48	.....	.....	....	1 220	13	.....	.....	....
1913– 14	16 200	116	16 200	32 200	199	11 100	139	11 100	13 300	120	13 900	144	13 900	18 200	131
1914– 15	9 310	67	9 310	11 100	119	7 750	97	6 440	8 290	129	5 920	61	5 920	8 200	138
1915– 16	11 200	81	11 200	14 200	127	8 040	100	8 040	10 700	133	8 920	92	8 920	10 400	117
1916– 17	13 200	95	13 200	17 400	132	7 550	94	.....	.....	....	13 500	140	13 500	22 900	170
1917– 18	11 800	85	11 800	14 300	121	6 940	86	6 940	8 620	124	10 800	111	10 800	11 900	110
1918– 19	7 740	56	7 740	9 700	125	7 060	88	7 060	7 540	107	13 100	135	13 100	22 000	168
1919– 20	6 800	49	6 500	8 860	130	5 500	69	5 500	6 820	124	3 210	33	3 210	3 700	115
1920– 21	10 900	78	10 900	16 200	149	7 350	91	7 350	11 100	150	11 500	119	11 500	20 600	179

TABLE 2.—Comparison of Geography and Drainage for California Streams

	Kern River, near Bakersfield, Calif.	Tule River, near Porterville, Calif.	Kaweah River, near Three Rivers, Calif.	Kings River, near Sanger, Calif.	San Joaquin River, near Friant, Calif.	Fresno River, near Knowles, Calif.	Merced River, near Exchequer, Calif.	Tuolumne River, near La Grange, Calif.	Stanislaus River, near Knight's Ferry, Calif.	Mokelumne River, near Clements, Calif.	Cosumnes River, near Michigan Bar, Calif.
Drainage area, in square miles	2 345	266	520	1 740	1 640	134.4	1 020	1 542	987	631	525
Average elevation, in feet.	6 780	4 185	5 805	7 195	7 050	3 215	5 570	5 892	5 055	5 020	3 200
Distance from Sanger, in miles	100 (S.)	58 (S.)	38 (S.)	.....	22 (N.)	36 (N.)	73 (N.)	85 (N.)	100 (N.)	135 (N.)	155 (N.)
Maximum 24-hour flood, in second-feet	16 125	5 430	10 100	43 930	38 800	3 770	37 200	52 600	36 900	16 700	22 400
Second-feet per square mile	6.88	20.41	19.42	25.25	23.66	28.07	36.47	34.11	37.39	26.47	42.67
Mean 24-hour flood, in second-feet	5 093	2 556	4 683	17 358	17 118	1 807	13 178	19 329	13 903	8 036	9 696
Second-feet per square mile	2.17	9.61	9.01	9.98	10.44	13.45	12.92	12.54	14.09	12.74	18.47

Keeping in mind the relation of mean run-off on the eleven streams, a maximum 24-hour flood was determined for the Kings River, at Sanger, which corresponded to the highest 24-hour flow on each stream in turn. The fact that the flood flow on each stream was the maximum for a period varying from 8 to 26 years had the effect of giving many comparable data for the Kings River at Sanger. Although the maximum rate of run-off is usually more important than the average for 24 hours, the 24-hour flood records were used because they were greater in number.

When the maximum 24-hour flood, in second-feet per square mile, for each stream was plotted according to its geographic location northwest or southeast of the Kings River (Fig. 7), a variation was shown similar to that for precipitation (Fig. 2). Flood flows decreased rapidly to the southward. The relative humidity of a storm, as it travels southward, becomes less, and the Kern Basin receives only slight precipitation from storms that cause intense run-offs on other streams farther north. The highest flows on the Kern River are usually caused by melting snow in May and June.

Geographic location is also of major importance in the other causes of flood variations that were considered. The average elevation receiving the heaviest precipitation is shown by Fig. 8. It is seen to be about 5 500 ft. The Cosumnes River has an average elevation of only 3 200 ft., but as it is at the north end of the area studied, it receives a high precipitation, which produces an extreme flood run-off. The Tule River, on the other hand, is of greater average elevation, but as it is near the south end of the valley the precipitation and flood yield is much smaller.

In relation to size of drainage area small floods were produced by the small secondary streams, the Fresno and the Tule, and by the Kern, a primary stream (Fig. 9).

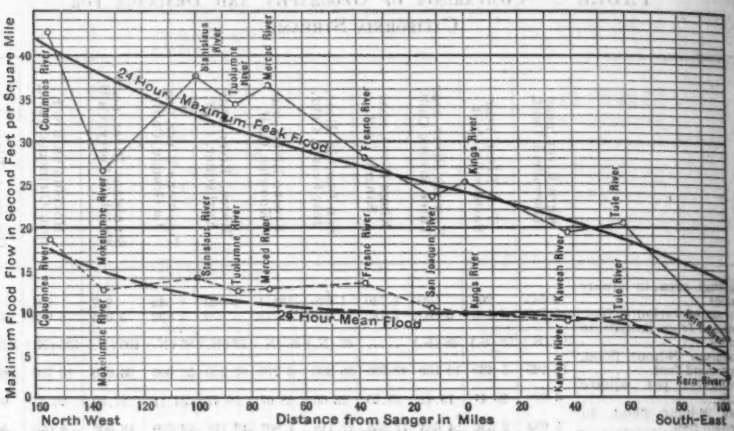

FIG. 7.—RELATION OF FLOOD RUN-OFF (24-HOUR) TO GEOGRAPHICAL LOCATION OF ELEVEN CALIFORNIA STREAMS.

In addition to the three factors of geographic location, average elevation, and drainage area, a fourth factor, the variation in proportion of the maximum 24-hour flood to the arithmetical mean, was used. This factor represents the susceptibility of a drainage area to floods or excessive rains. When a basin has a low variation factor the high stage will usually be caused by the sustained flows of melting snow in early summer; this is comparatively constant from year to year. High variations result from heavy storms during the early part of winter. Other factors, such as geologic formation, general topography, shape of the drainage basins, etc., are in this case of minor importance for, as has been pointed out, these physical features are similar among drainage basins in this area.

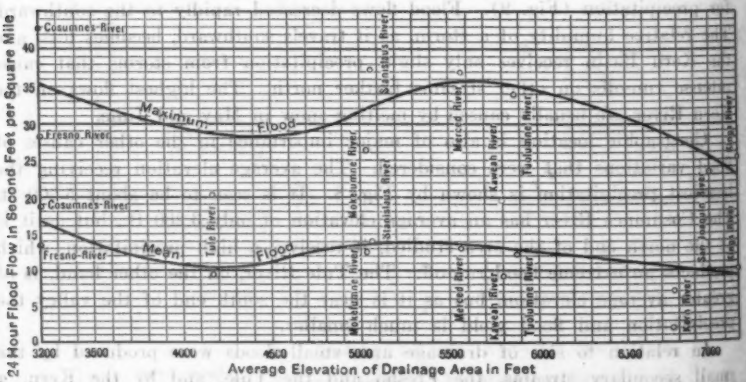

FIG. 8.—RELATION OF FLOOD RUN-OFF (24-HOUR) TO AVERAGE ELEVATION OF DRAINAGE AREA OF ELEVEN CALIFORNIA STREAMS.

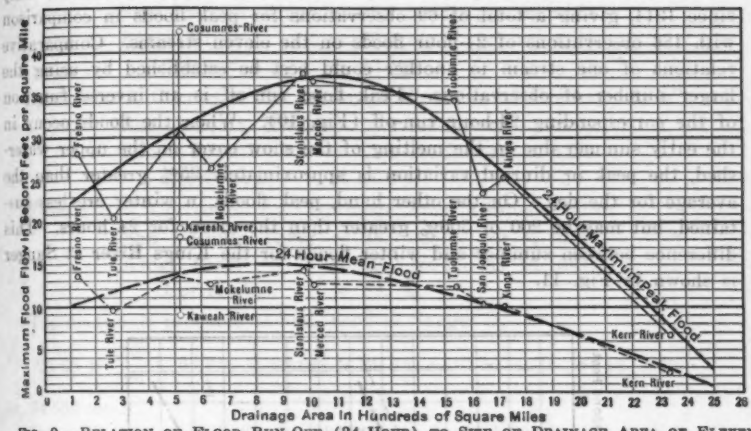

FIG. 9.—RELATION OF FLOOD RUN-OFF (24-HOUR) TO SIZE OF DRAINAGE AREA OF ELEVEN CALIFORNIA STREAMS.

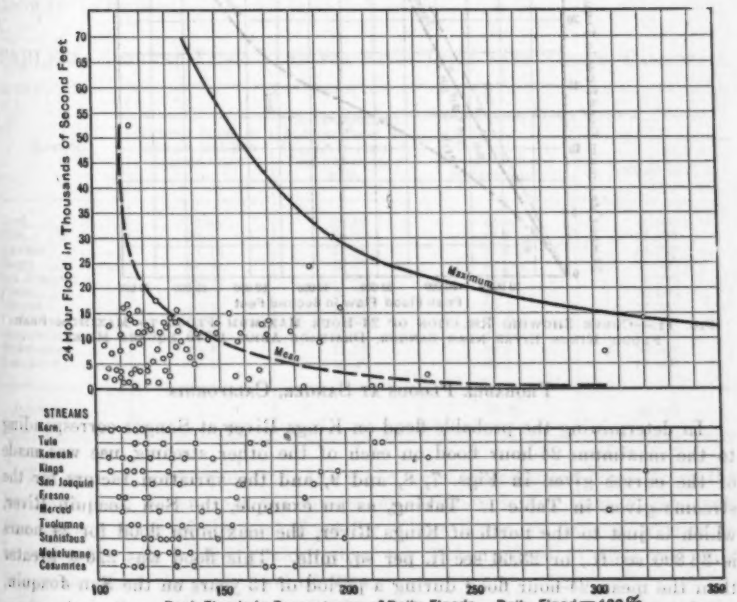

FIG. 10.—RELATION OF PEAK FLOOD TO 24-HOUR FLOOD FOR ELEVEN CALIFORNIA STREAMS.

Relation of Peak Flood to 24-Hour Flood

Peak floods have been published for the streams under consideration only since 1914, giving a total of 69 observations for peak floods in comparison with 186 observations of 24-hour floods on the eleven streams. Comparative relations of one stream to another could best be established by using the larger number of observations. Peak flood run-off is an inverse function of the corresponding 24-hour run-off (Fig. 10). When the floods occur in the early summer due to the melting of the snow cover on the upper watershed, the peak or diurnal variation is approximately 25% greater than the average for the day. On the other hand, peak floods in winter are less sustained, but may be 200 or 300% greater than the mean for 24 hours. This difference between summer and winter floods for the Kings River at Sanger is shown in Fig. 11.

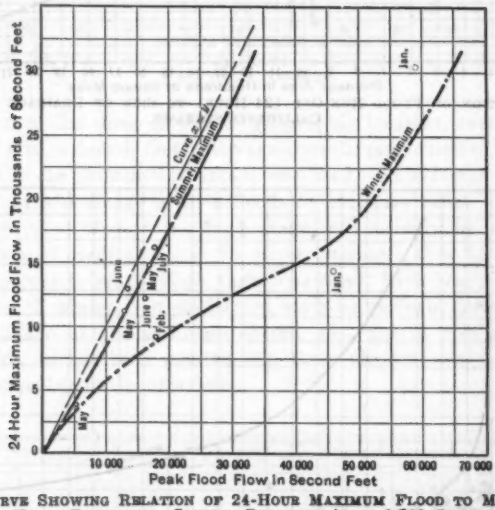

Fig. 11.—Curve Showing Relation of 24-Hour Maximum Flood to Maximum Peak Flood, Kings River near Sanger, Drainage Area, 1 740 Square Miles.

Probable Floods at Sanger, California

In determining the probable flood on Kings River at Sanger corresponding to the maximum 24-hour flood on each of the other streams, use was made of the curves given in Figs. 7, 8, and 9, and the variation factors for the streams given in Table 1. Taking, as an example, the San Joaquin River, which is just to the north of Kings River, the maximum flood for 24 hours is 38 800 sec-ft., or 23.66 sec-ft. per sq. mile. This flood was 226% greater than the mean 24-hour flood during a period of 13 years on the San Joaquin, while the maximum variation on Kings River was 253% greater than the mean.

In addition:

From Fig. 7, Correction for "Distance from Sanger":

San Joaquin River = 25.6 sec-ft. per sq. mile.
Kings River = 23.9 sec-ft. per sq. mile.
Correction = 93.3%

From Fig. 8, Correction for "Average Elevation of Drainage Area":

San Joaquin River = 25.8 sec-ft. per sq. mile.
Kings River = 23.2 sec-ft. per sq. mile.
Correction = 90.0%

From Fig. 9, Correction for "Drainage Area in Square Miles":

San Joaquin River = 28.0 sec-ft. per sq. mile.
Kings River = 25.0 sec-ft. per sq. mile.
Correction = 89.3%

The maximum probable 24-hour flood at Sanger is,

$$23.66 \times \frac{253}{226} \times 93.3\% \times 90.0\% \times 89.3\% = 19.85 \text{ sec-ft. per sq. mile}$$

or, 34 550 sec-ft.

From Fig. 10, a peak flood of 64 200 sec-ft., or 36.90 sec-ft. per sq. mile, would be possible from a 24-hour flood of 34 550 sec-ft.

The other ten streams were studied in a like manner and gave the results shown in Table 3.

TABLE 3.—Deduced Flood Flow for Kings River near Sanger, California.

Stream.	24-hour flood, in second-feet per square mile.	Corresponding 24-hour flood, at Sanger, in second-feet per square mile.	Peak Flood, Kings River at Sanger.	
			Second-feet per square mile.	Second-feet.
Kern	6.88	25.10	42.6	74 100
Tule	20.41	25.25	42.6	74 200
Kaweah	19.42	14.28	31.5	54 900
Kings	25.25	25.25	42.2	73 300
San Joaquin	28.66	19.85	36.9	64 200
Fresno	28.07	22.10	41.4	72 100
Merced	26.47	11.77	30.0	52 200
Tuolumne	34.11	13.58	32.0	55 500
Stanislaus	27.39	13.20	31.1	54 100
Mokelumne	26.47	11.83	30.2	52 500
Cosumnes	42.67	15.15	33.1	57 500
Average	.....	.....	35.78	62 240

The greatest variations from the mean were the adjusted floods from the Kern and Tule Rivers, but the maximum variation is less than 20 per cent. The average value of the peak flood for Kings River, at Sanger, appears to be reasonable. It is 42% greater than the 24-hour flood of 1901 and only 4% greater than the peak flood of 1914.

As a check, a maximum probable flood was computed for each of the other ten streams in the same manner as for the Kings River. In each case the computed flood was reasonable.

Probable Floods at Balch Camp, California

The original intention of this investigation being to determine the probable maximum flood on the North Fork of Kings River, studies were made to determine the probable flow at Balch Camp at the confluence of the North and West Forks of Kings River for a given flow at Sanger on the main river. The San Joaquin Light and Power Corporation has planned a series of developments on the North Fork of Kings River (Fig. 12). The first stage of the Balch Project has been constructed (1927). An arched diversion dam was built at Williams Crossing which will have an ultimate crest elevation of 4 110 ft. The water is then led by a 19 500-ft. tunnel along the north side of the stream to the penstock. Owing to the steepness of the canyon there was no feasible power-house site on the north side of the river, and the penstock was carried over the river to the station on the opposite side.

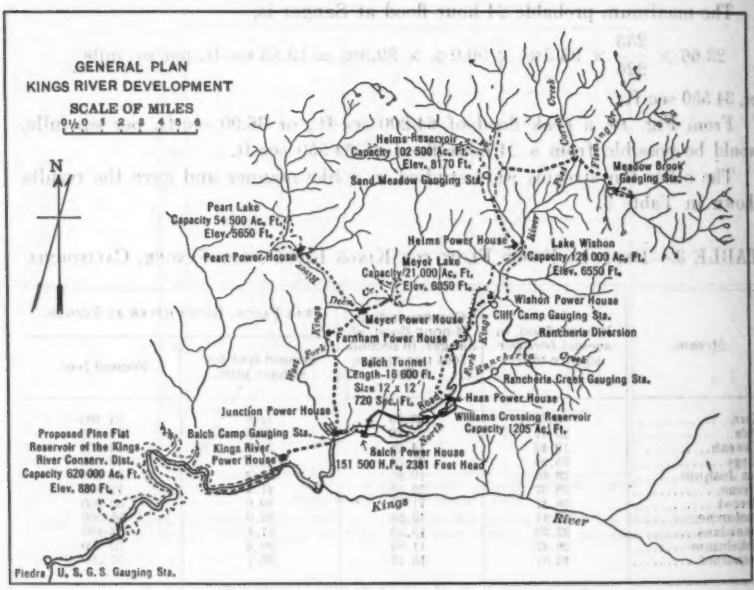

Fig. 12.

Four gauging stations (Fig. 12) were established on the North Fork water-shed in 1920: Balch Camp (1 240 ft.); Cliff Camp (6 150 ft.); Helms Creek (called Sand Meadow) (8 020 ft.); and Meadow Brook (8 140 ft.). In 1924 a fifth station was established on Rancheria Creek (6 400 ft.). On the West Fork there are two stations, near Balch Camp (1 310 ft.), and Dinkey Meadows (5 440 ft.). These stations have been maintained in co-operation with the U. S. Geological Survey. Flood run-off relations, which might be called secondary relations in contrast to the primary relations existing between the eleven different water-sheds, were deter-

mined for the main Kings River Station at Sanger and the North Fork Stations. These relations were expressed in second-feet per square mile for varying average elevations of water-shed (Fig. 13) and in square miles for drainage areas (Fig. 14). As there have been few floods on the North Fork since the stations were established, it was necessary to consider only the corresponding term of years at Sanger. Therefore, as more data are made available the run-off per square mile will be increased, but the relative value

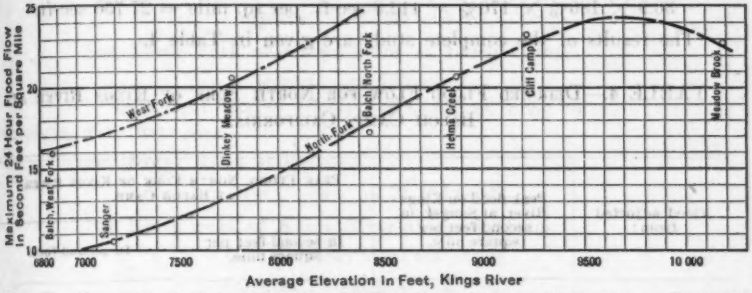

FIG. 13.—RELATION OF FLOOD RUN-OFF (24-HOUR) TO AVERAGE ELEVATION OF DRAINAGE AREA OF KINGS RIVER, CALIFORNIA.

from station to station should hold true. Insufficient data were at hand to determine variation factors for the North Fork Stations. As the relative difference in factors which determine run-off, such as slope, exposure to storm, forest and vegetable cover, etc., was small between Sanger and Balch Camp, the difference in variation factors should be slight. However, if the probable flood at an upper station, as Cliff Camp, were being estimated, it would be necessary to take the ratio of variation factors into consideration.

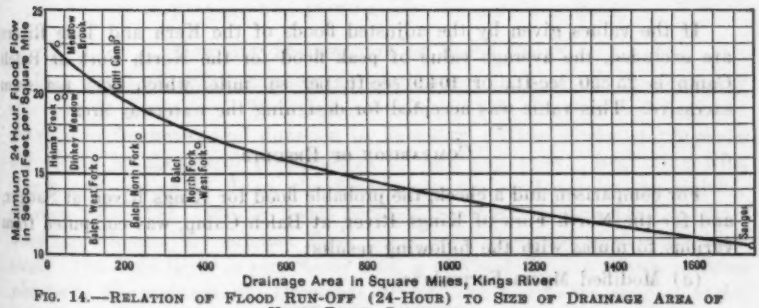

FIG. 14.—RELATION OF FLOOD RUN-OFF (24-HOUR) TO SIZE OF DRAINAGE AREA OF KINGS RIVER, CALIFORNIA.

The method followed in determining the maximum peak flood on the North Fork at Balch Camp was similar to that used in determining the primary floods at Sanger, as follows:

From Fig. 13, Correction for "Average Elevation of Drainage Area":

Kings River at Sanger	= 10.4 sec-ft. per sq. mile.
North Fork at Balch Camp	= 17.7 sec-ft. per sq. mile.
Correction	= 170%

From Fig. 14, Correction for "Drainage Area in Square Miles":

Kings River at Sanger	= 10.4 sec-ft. per sq. mile.
North Fork at Balch Camp	= 18.7 sec-ft. per sq. mile.
Correction	= 180%

The maximum 24-hour flood on the San Joaquin adjusted to suit Kings River conditions produced a peak flood of 36.9 sec-ft. per sq. mile at Sanger. The corresponding peak at Balch Camp would be:

$$36.9 \times 180\% \times 170\% = 112.8 \text{ sec-ft. per sq. mile} = 27\,750 \text{ sec-ft.}$$

The results of the complete study are given in Table 4.

TABLE 4.—DEDUCED FLOOD FLOW FOR NORTH FORK OF KINGS RIVER AT BALCH CAMP, CALIFORNIA.

Flood adjusted from:	Peak flood on Kings River, at Sanger, in second-feet per square mile.	PEAK FLOOD, NORTH FORK OF KINGS RIVER, AT BALCH CAMP.	
		In second-feet per square mile.	In second-feet.
Kern	42.6	130.0	32 000
Tule	42.6	130.0	32 000
Kaweah	31.5	96.5	23 800
Kings	42.2	129.0	31 800
San Joaquin	36.9	112.8	27 750
Fresno	41.4	126.8	31 200
Merced	30.0	91.9	22 700
Tuolumne	32.0	97.8	24 100
Stanislaus	31.1	95.2	23 500
Mokelumne	30.2	92.5	22 800
Cosumnes	33.1	101.2	25 000
Average	35.78	109.5	26 970

If the values given by the adjusted floods of the Kern and Tule Rivers are excluded, the average value of peak flood for the North Fork at Balch Camp is 25 850 sec-ft., or 104.9 sec-ft. per sq. mile, which does not seem excessive. This value was accepted for designing the waterway area.

COMPARISON OF RESULTS

For comparison and a check, the probable flood for Kings River, at Sanger, and for the North Fork of Kings River, at Balch Camp, was computed from various formulas with the following results:

(*a*) Modified Myers Formula*:

$$Q = C\sqrt{M}$$

in which, M is the drainage area, in square miles.

For interior drainage of California,

$$C = 1\,860$$

For Kings River at Sanger,

$$Q = 1\,860\sqrt{1\,740} = 77\,500 \text{ sec-ft.}$$

* "Flood Flow Characteristics," by C. S. Jarvis, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 89 (1926), p. 985.

For North Fork of Kings River at Balch Camp,

$$Q = 1\,860\sqrt{246.4} = 29\,150 \text{ sec-ft.}$$

(*b*) Fuller Formula:*

$$Q(\text{ave.}) = C\,A^{0.8}$$

in which, $C = 70$.

For Kings River at Sanger,

$$Q(\text{ave.}) = 27\,400 \text{ sec-ft.}$$

For North Fork of Kings River at Balch Camp,

$$Q(\text{ave.}) = 5\,735 \text{ sec-ft.}$$

$$Q(\text{24-hour flood in } T\text{-years}) = C\,A^{0.8}\,(1 + 0.8 \log T)$$

in which, $T = 100$ years.

For Kings River at Sanger,

$$Q(\text{in } T\text{-years}) = 71\,300 \text{ sec-ft.}$$

For North Fork of Kings River at Balch Camp,

$$Q(\text{in } T\text{-years}) = 14\,920 \text{ sec-ft.}$$

$$Q(\text{max.}) = Q(1 + 2\,A^{-0.3})$$

in which Q = 24-hour flood in T-years.

For Kings River at Sanger,

$$Q(\text{max.}) = 86\,500 \text{ sec-ft.}$$

For North Fork of Kings River at Balch Camp,

$$Q(\text{max.}) = 20\,650 \text{ sec-ft.}$$

(*c*) *Bulletin No. 5*, "Flow in California Streams", Department of Public Works, California:

For Kings River, at Sanger,

($T = 100$ years) = 41 000 sec-ft. (for a 24-hour flood)

Monthly Distribution of Flood Peaks

Peak floods on California streams of the west slope of the Sierras never occur before December. They are most frequent in January and begin to decrease in frequency and duration in February. These winter storm peaks are caused by the direct precipitation of rain and snow. Beginning with March and ending in June, the flood peaks are produced by the melting of snow (Fig. 15). Winter floods produce a sudden extreme peak of short duration, whereas the summer flood has a long sustained moderately high stage. The Tule River is very flashy and, as is shown by Fig. 15(*a*), in three months of the peak season—December, March, and April—the greatest variation of the peak flood from the 24-hour flood occurred on the Tule. However, the greatest variation, 320%, which was produced by a winter storm in January, took place on the Kings River.

Expectancy of Flood Occurrence

The fact that flood run-off above the mean is more variable than flood flow below the mean has been long established. Rainfall follows the same

* "Flood Flows," by Weston E. Fuller, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXVII (1914), p. 564.

rule. Flood records for any stations covering a period of years will show a few great floods of high variation from the mean and several floods near or slightly below the average. The length of record on any of the California

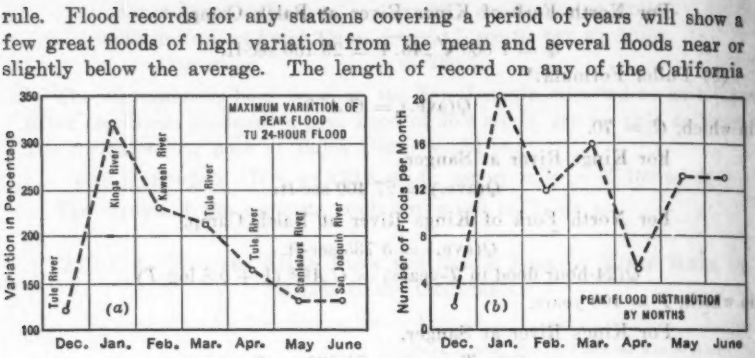

FIG. 15.—FREQUENCY AND INTENSITY OF PEAK FLOODS, ON ELEVEN CENTRAL CALIFORNIA STREAMS.

streams is too short to make an accurate forecast of the frequency of maximum floods. A graphical analysis of the frequency of Kings River floods at Sanger is given in Fig. 16, the data used being shown in Table 5. By

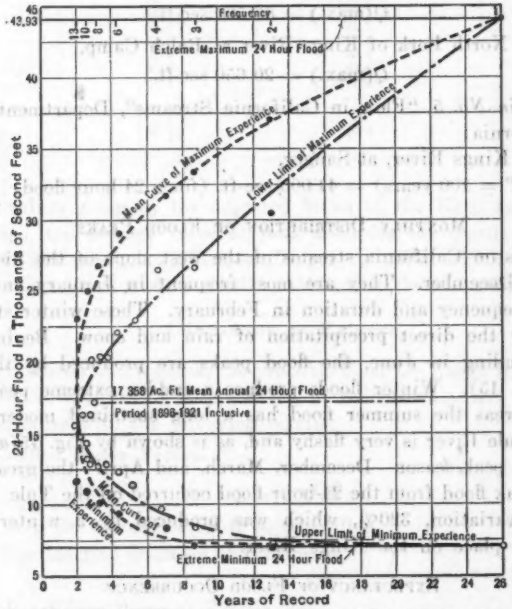

FIG. 16.—FLOOD FREQUENCY ON KINGS RIVER, NEAR SANGER, CALIFORNIA.

prolonging this curve in both directions it is seen that greater and lesser floods may be expected when a greater number of records are available. No

trustworthy estimate is possible by this method, however, for judgment would indicate that both maximum and minimum curves should become parallel with the base when the most extreme conditions are reached; but what those extreme conditions are, is problematic.

TABLE 5.—EXPERIENCE TABLE OF ANNUAL 24-HOUR FLOOD AT PIEDRA, CALIFORNIA.

Order of magnitude.	FLOODS IN ORDER OF MAGNITUDE, IN ACRE-FEET.		Number of annual floods averaged.	MEAN ANNUAL FLOODS, IN ACRE-FEET.	
	Maximum.	Minimum.		Maximum.	Minimum.
1	43 930	6 920	1	43 930	6 920
2	30 400	7 210	2	37 165	7 065
3	26 600	8 348	3	33 643	7 493
4	26 380	9 795	4	31 827	8 068
6	22 100	12 400	6	28 690	9 312
8	20 800	12 800	8	26 618	10 172
10	17 290	13 200	10	25 043	10 737
13	15 700	15 600	13	22 979	11 659

STORAGE

On a majority of the eleven streams considered, reservoirs of relatively large capacity have been constructed. These reservoirs are in the foothills and their primary use will be for irrigation. Supplemented by storage for power purposes near the head-waters, they will have some favorable effect on the flood flow on the tributary rivers. Being constructed principally for irrigation, a reservoir might be full when a great flood comes; nevertheless, to a certain extent, it would elongate the flood wave and thereby reduce the peak discharge. It would be a great community benefit if a certain proportion of the total capacity could be held for flood protection until the flood menace of a high year was past. This is practiced with a few of the larger U. S. Reclamation Service reservoirs. Although the principal effect of such reservoirs will be to reduce the flood peaks, they will probably have comparatively little effect on the maximum discharge of the extreme flood, which is expected only once or twice in a century.*

FLOOD ESTIMATES

Floods on any water-shed are caused by a storm of a certain type, which varies but little from year to year. If flood records of at least 100 years were available for each stream, it is evident, of course, that the best way to investigate the variations of flood discharge would be to treat each stream by itself. However, on many streams, few or no records are at hand. A fair estimate may be had by adjusting the recorded discharges of neighboring streams, similar to the stream in question in geologic, topographic, and meteorologic characteristics. The adjusting factors may not be the same in

* The relation of these foothill irrigation reservoirs to power development is discussed by William Kelly, M. Am. Soc. C. E., in his paper entitled "Co-Ordination of Irrigation and Power," *Transactions*, Am. Soc. C. E., Vol. 89 (1926), p. 1429.

all cases, but size of drainage area will be a persistent influence. This comparison or adjustment of existing flood records should be supplemented by the solution of reliable flood formulas, by a study of rainfall records by the time-area-depth method, by reliable evidence of flood deposits, as drift and alluvial bars, and the experiences of observers. Precipitation records, especially if the precipitation is in the form of rain, will serve as a basis of flood prediction, under ideal conditions. Under actual conditions a given rainfall intensity will yield extreme variations of run-off.

Conclusions

In this paper, the writer does not assume to solve the wide problem of flood flows, but merely to describe one method of attack. The future can best be predicted by a study of the past. A rational study of past flood records should form the basis of estimating probable future floods on any stream. The full purpose will be accomplished when a better appreciation of flood danger is held by the Engineering Profession and by the general public. Only after this knowledge is general will the proper measures of flood control be taken.

In addition to the specific acknowledgments made in the paper, the writer has obtained valuable data from the published writings of many engineers, most of them members of this Society. This study was made for the San Joaquin Light and Power Corporation, under the general direction of Mr. J. W. Jourdan. H. K. Fox, M. Am. Soc. C. E., Construction Superintendent, was in charge of the Kings River Development.

DISCUSSION

ROBERT FOLLANSBEE,* M. AM. SOC. C. E. (by letter).—The author states that, up to a certain elevation, ranging from 4 000 to 6 000 ft., precipitation increases with height, but higher than that, it decreases slowly to the summit. That this statement concerning a zone of maximum precipitation below the summit is not applicable to the Rocky Mountain region is shown by precipitation records in Colorado, so situated as to be comparable.

TABLE 6.—PRECIPITATION RECORDS OF NORTHERN COLORADO.

Slope.	Station.	Elevation, in feet.	Mean precipitation, in inches.	Length of record, in years.
West	Fraser	8 560	19.12	8
East	Longs Peak (near)	8 600	20.99	9
West	Spruce Lodge	9 600	30.13	6
East	Silver Lake	10 500	37.03	10
Summit	Corona	11 600	45.99	6

The first set presented comprises stations located on a section through the Continental Divide in Northern Colorado, on both eastern and western slopes, with one station at the summit. These records are summarized in Table 6.

The second set is located near a section extending from Colorado Springs at the base of the mountains to the summit of Pike's Peak (Table 7).

TABLE 7.—PRECIPITATION RECORDS IN COLORADO.

Station.	Elevation, in feet.	Mean precipitation, in inches.	Length of record, in years.
Colorado Springs	6 098	14.37	33
Glen Eyre	6 500	16.15	18
Fremont Experiment Station	8 850	22.79	10
Lake Moraine	10 265	24.98	24
Pike's Peak	14 107	29.55	15

Even a casual comparison of the two sets will show that elevation is not the only factor in determining the amount of precipitation, since that on Pike's Peak at 14 107 ft. is less than the precipitation at 9 600 ft. on the Continental Divide. The writer has shown elsewhere† that the controlling factor appears to be the presence or absence of mountain masses to the westward.

HAROLD K. FOX,‡ M. AM. SOC. C. E. (by letter).—The water records of California streams do not cover long periods. The best and oldest records have been taken at points where the streams enter the valleys (usually above irrigation intakes), and were primarily established for purposes of irrigation. In investigating power resources, the several branches and tributaries in the high altitudes are the points of interest, and it is only recently that records

* Dist. Engr., U. S. Geological Survey, Denver, Colo.

† *Water Supply Paper 500-C*, U. S. Geological Survey, p. 69.

‡ Supt. of Constr., San Joaquin Light & Power Corporation, Fresno, Calif.

have been obtained on such streams. A large part of these records are for the period since about 1917 and, since the past ten years have been somewhat less than normal in precipitation, it is necessary to combine these records with those of longer duration on the main stream and to compare one river with another.

The rapid development of power has necessitated exhaustive studies of water resources and flood flow. The writer has a close personal knowledge of the vast amount of work that Mr. Reed has done in preparing this paper and hopes that it will be as beneficial to others who may have occasion to use it as it has been to those who developed the Kings River Project of the San Joaquin Light and Power Corporation. On this project it was necessary to locate the first power house in a very narrow and deep canyon, and one of the problems was that of protection against floods.

The writer has consulted the author on the matter of flood determination and finds that his method has been changed in certain details. The primary data are now used to determine one relation curve, which is called the "geographical location" curve. The location of a drainage area in relation to the path of cyclonic storms is of primary importance in this section of California.

The relation of average elevations and size of drainage area are, however, very important. Referring to Fig. 3, it will be noted that the elevation of maximum precipitation from the normal storm is about 6 000 ft. for Kings River. Storms which produce the highest run-off on the Kings River occur in January and February. At this season precipitation on the upper part of the water-shed is in the form of snow and contributes only a small run-off. Above Elevation 7 000 the mean monthly temperature is below 30° Fahr., in January and February. During some days of these months the temperature would be above freezing, but the contribution to the run-off would be slight. Thus, the effective area would be much reduced. The effective area of the Kings River above Sanger would be about 900 sq. miles instead of 1 740 sq. miles.

The mean elevation of the effective area is then determined. The various streams will then be compared with respect to the relative location of the mean elevation of their effective areas. If the mean elevation coincides with the point of maximum precipitation, it will have a factor of 1.0. On the other hand, if it falls above or below the elevation of peak precipitation, the factor will be suitably lowered. In this manner, factors will be obtained for evaluating the effect of average elevation.

The observation that a stream of small tributary area will have a higher run-off per square mile than a large stream from a storm of given intensity has been the basis of many flood formulas. The effective area at the time of maximum flood expectancy should again be used. In order to eliminate the effect of difference in precipitation, only four streams will be used in determining this curve—the Kings, Kaweah, San Joaquin, and Merced Rivers.

In February, 1927, a storm occurred on the Kings River which produced a run-off of high intensity. The locus of the storm was over a relatively small area about five miles west of the junction of the North Fork with the main stream, or about eight miles from Balch Camp. The area had been burned

over by a forest fire during the previous July. Periods of moderate rainfall preceded the main storm, which was of cloudburst intensity. Each small watercourse became a torrent and, since there was no grass or underbrush to hold the soil, erosion was very severe. Each watercourse became a gully, and boulders (many 10 ft. in diameter) were washed down, filling the channel of Kings River and forming pools of slack water. These dams sustained the flow very well.

The gauge at Piedra showed a peak discharge from this storm of about 22 000 cu. ft. per sec. This discharge from the upper reaches of the stream was not high; the discharge of the North Fork at Balch Camp was about 2 000 cu. ft. per sec. A conservative estimate of the run-off from the main storm area is 10 000 cu. ft. per sec. from approximately 100 sq. miles. This storm area was in the foothills, where there are moderate slopes.

It is not improbable that at some future date the locus of a similar storm will be over the North Fork area, where the run-off would be even more rapid, due to the steeper slopes and less cover. If that should happen, a run-off of 25 000 sec-ft. (or 101.5 cu. ft. per sec. per sq. mile) is entirely possible. A storm of this intensity will probably never occur before December, when the upper water-shed would contribute little to the flow. The run-off from the effective area might then be 150 cu. ft. per sec. per sq. mile.

CHARLES H. LEE,* M. AM. SOC. C. E. (by letter).—This paper describes a method of estimating maximum flood discharge which assumes that within the same meteorological province the records of several streams, similar in storage, vegetation, soil, and other physical characteristics, can be combined, after making certain adjustments, so as to give an equivalent record for one of the streams. The adjustments include corrections for geographic position, average elevation and size of drainage area, and relation of peak to 24-hour flood discharge. The author states that he adopts this method because of the inherent limitations of flood discharge formulas and the impracticability of extending their use to streams other than those for which they were originally devised.

The writer's experience in estimating flood discharges from formulas confirms these conclusions, but has led him to favor the rational method rather than the stream-group method. Drainage areas have individual characteristics. These are much more effective in producing differences in flood discharge than they are in the case of total run-off. With certain modifications, the author's method might be applied to the eleven streams which he studied, if his purpose was to determine long-time, mean, total run-off, but it is not logically applicable to the problem of determining flood discharge.

Throughout the paper the writer detects a confusion of thought regarding factors which influence total run-off and maximum flood run-off. It is believed that a rational analysis of these factors may be helpful.

* Cons. Hydr. Engr., San Francisco, Calif.

Total Run-Off.—The total run-off from a drainage area is usually considered for an annual cycle (or water-year) and depends on the total precipitation on the drainage area during the period, minus losses due to evaporation from water, snow, ice, and soil, and transpiration from plants. Cyclic surface, snow, or ground storage may be important if wide variation occurs in precipitation from year to year. Total run-off may be expressed for unit area by the formula, $r = P - L \pm S$, in which, r is the total run-off from a unit area; P, the depth of annual precipitation; L, the depth of annual losses; and S, the annual storage correction.

Water, snow, and ice evaporation depend on the extent and period of exposure of the respective surfaces, and the temperature, humidity, and other factors which influence the evaporation rate. Elevation and geographic location as they affect meteorological conditions play an important part in fixing these losses. Soil evaporation depends on slope, quantity, and time distribution of rainfall, vegetation, depth, and porosity of the soil, and the factors which influence the rate of water evaporation. Transpiration depends on the type and extent of vegetation, which, in turn, depends on soil, rainfall, and elevation and geographic location as they affect meteorological conditions. Cyclic storage ordinarily becomes important only when artificial surface storage of large capacity has been constructed, or large ground storage capacity exists in the form of gravel beds or volcanic débris.

Flood Discharge.—In contrast to total run-off, this factor depends on the intensity of rainfall and the amount of run-off finding its way immediately into drainage channels. It may be expressed for a unit area by the formula, $q = Ci$, in which, q is the discharge, in second-feet, for a unit area; i, the intensity of rainfall, in inches per hour; and C, the run-off coefficient representing the ratio of the rate of immediate run-off to rainfall intensity and expressing the physical characteristics of the drainage area.

As the automatic rainfall records obtained during the past 20 to 30 years have demonstrated, rainfall intensity at any point varies greatly with time, being greatest for periods of a few minutes, decreasing rapidly for longer periods, and approaching a more constant value for periods exceeding 12 hours. Maximum flow at any point on the stream results from a rainfall the duration of which equals or exceeds the time of concentration of water from the most remote point (in time) on the drainage area. It is greatest when the surface is frozen or saturated from a previous storm and may be augmented in special cases by the effect of warm rain in melting snow on the ground.

The maximum rainfall intensity to be considered for a given drainage area is not uniform within the same "meteorological province", but varies from place to place depending on local conditions and especially on the time of concentration. The latter differs for every point on the stream. It may also differ at the same point for different storms, depending on the extent of the area upon which precipitation falls in the form of rain. For example, precipitation might fall in the form of snow on the higher portion of the drainage area, or the maximum storm might not extend over the whole of the area.

The time of concentration is controlled by the following factors:

1.—The time of flow over the surface of the ground to the nearest stream channel. This is influenced by the average slope of the ground at the most distant point in the drainage area and the irregularities of the surface produced by vegetation and character and condition of the soil surface.

2.—The time of flow in stream channels. This is determined by the average slope and character of channel, as reflected in the value of n in Kutter's formula.

3.—The shape of the drainage area, whether long and narrow, fan-shaped, etc.

4.—The arrangement of branches and laterals.

5.—Surface storage.

The percentage of rainfall which finds its way immediately into the stream depends on:

(*a*) Losses, consisting of immediate evaporation (very small) and absorption. The latter is controlled by surface conditions, such as the extent of bare rock or frozen ground, the character of the soil, the slope, and the existence of a mat of vegetable material and humus.

(*b*) Temporary retention due to storage in irregularities of the surface and capillary action, the vegetation mat being an important factor in the matter.

(*c*) The degree of previous saturation, depending on the season of the year when the storm occurs and the relation of the time of the maximum storm to the storm period.

Table 8 summarizes these factors and serves to contrast the differences between those entering into total run-off and maximum flood run-off.

TABLE 8.—COMPARISON OF PRINCIPAL FACTORS CONTROLLING TOTAL RUN-OFF AND MAXIMUM FLOOD RUN-OFF FOR A GIVEN DRAINAGE BASIN OF AREA A.

Total run-off, $R = (P - L \pm S)\,A$.	Maximum flood run-off, $Q = C\,i\,A$.
A = superficial area of whole drainage basin.	A = portion of superficial drainage area covered by storm yielding rainfall of maximum intensity.
P = total depth of precipitation during complete run-off cycle (water-year).	i = maximum rate of rainfall of uniform intensity continuing throughout the period of concentration, or for the period of the storm if shorter than the period of concentration.
$L \pm S$ = depth of accumulated losses during the entire water-year by evaporation from water, snow, ice, and soil, and transpiration augmented or diminished by cyclic surface or ground storage.	$1 - C$ = percentage of rainfall at maximum rate lost by absorption and temporary storage in surface irregularities and capillary films.
	C = percentage of rainfall at maximum rate which runs off immediately and appears in the stream by the end of the period of concentration.

In the light of this analysis (see Table 8), the following conclusions are believed to be sound:

First.—The author's implied assumption that the rainfall intensity producing maximum flood discharge is uniform throughout a meteorological province, or for all points on a drainage system, is not justified.

Second.—Few, if any, drainage areas (including their drainage systems) are so similar that both the times of concentration and the run-off factors for maximum discharge correspond, as the author's second assumption would imply.

Third.—The items selected by the author for making adjustments between various drainage areas have no direct or apparent relation to the factors which control maximum flood run-off.

In comment on the third conclusion, the distance north or south of Sanger, Calif., has not been shown to be systematically related to the form of the rainfall intensity curve at different points, nor to any factor influencing flood discharge. Also, the average elevation of the drainage area, although indicating the percentage of precipitation falling as snow, is not so expressive of this relation as would be the proportion of the drainage area above the critical elevation. Fig. 8 shows no definite relation between flood run-off and average elevation. Then, again, the size of the drainage area, although it is an important factor for extremely large areas, or in regions of local storms where a given storm covers only a portion of the area, has little bearing on flood discharge from Sierra Nevada streams. The latter are normally visited by general storms the extent of which far exceeds the limits of the drainage area. Fig. 9 shows clearly the absence of any relation between drainage area and maximum flood flow for these streams.

The apparent conclusion illustrated in Table 3 is that the peak flood discharge to be expected on Kings River at Sanger is 62 240 sec-ft. The maximum 24-hour flood reported for this station by the U. S. Geological Survey is 43 930 sec-ft. on January 7, 1901. Correcting this to peak flood by use of Fig. 10, a value of 73 800 sec-ft. is obtained, which exceeds the author's value by 18 per cent. Table 9 is obtained by assembling the various values given in the paper. It appears from this table that the author's value for peak flood on Kings River, at Sanger, is from 12 to 19% less than that derived by any other method. It is believed that the author's paper is incomplete without a discussion of the reasons which underlie the assumptions on which his method is based. This is especially true in view of the fact that it departs somewhat from accepted practice.

TABLE 9.—COMPARISON OF VARIOUS VALUES FOR PEAK FLOOD DISCHARGE OF KINGS RIVER, AT SANGER, CALIFORNIA.

Authority.	24-hour maximum flood, in second-feet.	Peak flood, in second-feet.
Modified Myers formula	.....	77 500
Fuller formula (100 years)	.....	71 300
Plate LXVIII, *Bulletin No. 5*, State of California, Department of Public Works (100 years)	41 000	70 520
Estimated flow, January 7, 1901, *Water Supply Paper 239*, U. S. Geological Survey	43 930	73 800*
Author (Table 3)	31 215	62 240

* Obtained from 24-hour flow by use of Fig. 10.

The rational method of estimating flood discharge was originally used in estimating maximum run-off from urban and suburban areas in connection

with storm-sewer design. The same method is applicable to natural drainage areas of sizes exceeding those considered by the author, although for very large areas its use is impracticable because of the complexity.

The general application of the rational method is becoming more and more feasible as the appropriate basic data are accumulated. Automatic tipping-bucket rainfall records kept by the U. S. Weather Bureau and other public agencies, are now available at many points throughout the United States. At many of these stations records covering from 15 to 25 years are available for 5-min. periods, and up to 50 years for periods of 12, 24, 36, and 48 hours, etc. From such data rainfall intensity curves can be constructed, which indicate the range of rainfall rates for various periods of time and frequency of occurrence. The normal expected intensities of rainfall at Oakland, Calif., within 2, 5, 10, and 15-year periods are as shown in Fig. 17. These curves are based on automatic recording rain-gauge observations covering 18 complete years of a 23-year record (1904 to 1927) at the Chabot Observatory. They are typical of the San Francisco Bay region. The same storms upon which these curves are based, extend over the western slope of the Sierra Nevada Mountains, including the drainage areas of the San Joaquin Valley streams considered by the author. In the absence of automatic rainfall records on the drainage area of Kings River and adjacent streams, the writer believes that this curve could be modified and extended from local foothill and mountain records for 12, 24, 36, and 48-hour periods so as to give a generally representative curve for this region, which would include all times of concentration that would occur in practice.

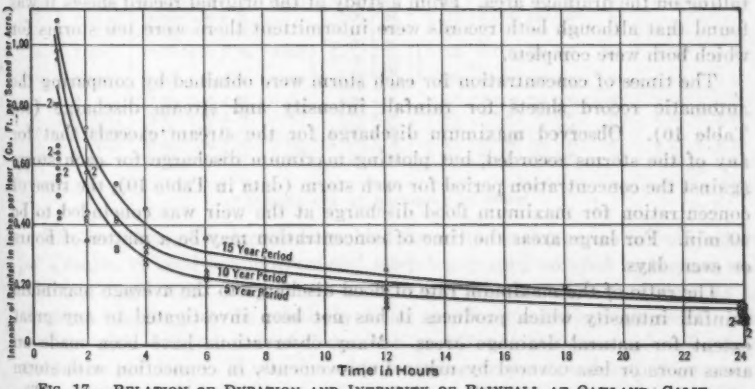

FIG. 17.—RELATION OF DURATION AND INTENSITY OF RAINFALL AT OAKLAND, CALIF.

The time of concentration at any point on a stream is readily estimated by observing the movement of flood waves down the channel, checked by velocity measurements during high water. It is to be recognized, however, that the water velocity usually indicates a more rapid rate of advance than the flood wave because the latter is modified by channel storage. The determination is most effectively made in conjunction with rainfall observations, but, on a long stream course, the observed rates of travel with proper modifications

may be extended back to stream head-waters. Time should also be allowed for the flow of water over the surface of the ground and down small tributaries before it reaches the larger channels.

The writer recently had an opportunity to determine the time of concentration for Strawberry Creek, a small stream flowing through the campus of the University of California at Berkeley and draining a canyon cut back into the hills on the east shore of San Francisco Bay. The area drained at the measuring weir is less than 1 sq. mile (509 acres) and has a range in elevation of from 1 200 to 1 500 ft. on the surrounding ridges to 500 ft. at the weir. The average distance from the ridge crests to definite stream channels is 0.25 mile, with a difference in elevation of from 400 to 600 ft. The greatest length of stream channel is 1.2 miles, with an average slope of 140 ft. per 1 000. The entire area supports a good growth of natural grass, with close-growing brush on the northern slopes and planting of young pines along the ridges at the head of the canyon.

Self-recording weir measurements of the flow were made by the University under the direction of Mr. H. B. Foster during the five seasons between 1910 and 1915. Automatically recorded rainfall records from a Friez tipping-bucket rain-gauge on the University Campus were also kept during the period by the College of Civil Engineering, under the direction of Charles Gilman Hyde, M. Am. Soc. C. E. During the season of 1914-15, comparative observations of rainfall on the University Campus and at thirteen rain-gauges placed at critical points on the Strawberry Creek drainage area indicated a practical agreement between the rainfall as measured at the campus and the average falling on the drainage area. From a study of the original record sheets it was found that although both records were intermittent there were ten storms for which both were complete.

The times of concentration for each storm were obtained by comparing the automatic record sheets for rainfall intensity and stream discharge (see Table 10). Observed maximum discharge for the stream exceeds that for any of the storms recorded, but plotting maximum discharge for each storm against the concentration period for each storm (data in Table 10), the time of concentration for maximum flood discharge at the weir was concluded to be 40 min. For large areas the time of concentration may be a matter of hours or even days.

The ratio of the maximum rate of flood discharge to the average maximum rainfall intensity which produces it has not been investigated to any great extent for natural drainage areas. Many observations have been made on areas more or less covered by urban improvements, in connection with storm sewer design; but run-off factors for such conditions do not apply. The factors generally used on such work for previous surfaces, such as parks, gardens, lawns, and meadows, vary from 0.05 to 0.25, depending on surface slope and character of subsoil.

Merrill M. Bernard, M. Am. Soc. C. E., states* that values for watersheds will vary from 0.11 to 0.15, but does not describe the types of drainage area to which such factors apply. It is probable that he has in mind the

* *Transactions*, Am. Soc. C. E., Vol. 89 (1926), p. 1087.

milder climates and flat slopes. Other authorities quote values of from 0.50 to 0.75, probably for colder regions and steeper slopes.* Mr. George Chamier gives the following values:†

Frozen ground	0.66
Naked unfissured mountains, Very steep ground, Paved streets	0.80
Mountainous and rocky country or non-absorbent surfaces	0.55 to 0.65
Wooded hill slopes and compact or stony ground	0.45 to 0.55
Meadows and gentle declivities, absorbent ground	0.35 to 0.45
Flat country, sandy soil, or cultivated land	0.25 to 0.35

TABLE 10.—Ratio of Maximum Rate of Flood Discharge to Maximum Intensity of Rainfall During Concentration Period for Strawberry Creek, Berkeley, California.

Storm number.	Date of storm.	Maximum Rate of Rainfall: Time.	Maximum Rate of Rainfall: Increase per hour.	Maximum Rate of Run-Off, in Second-Feet per Acre. Time.	Total.	Increase due to storm.	Maximum discharge, in gallons per hour.	Concentration period, in minutes.	Run-Off Ratio for. Total run-off.	Increase due to storm.
1	1/13/11	10:00–12:00 A.M.	0.070	1:30 P.M.	0.004	0.002	55 000	90	0.06*	0.04*
2	1/13/11	4:00– 5:00 P.M.	.....	6:00 P.M.	0.016	0.014	220 000	60	....	....
3	1/13/11	9:45–10:00 P.M.	0.178	11:00 P.M.	0.021	0.016	290 000	60	0.12	0.09
4	1/19/11	8:15–10:30 P.M.	0.200	10:00 P.M.	0.005	0.003	120 000	105	0.02*	0.01*
5	1/28/11	2:00– 4:00 A.M.	0.178	11:00 P.M. 3:00 A.M. 5:00 A.M.	0.021	0.020	285 000	60	0.12	0.11
6	3/ 1/11	2:30– 3:45 A.M.	0.110	4:30 A.M.	0.015	0.013	205 000	60	0.13	0.12
7	12/16/12	6:30– 7:00 A.M.	0.073	8:30 A.M.	0.001	0.001	20 000	90	0.02*	0.02*
8	1/18/13	6:15– 7:00 A.M.	.....	9:30 A.M.	0.009	0.007	120 000	150	....	....
9	1/12/14	10:30–11:00 A.M.	0.335	11:45 A.M.	0.051	0.039	700 000	45	0.15	0.12
10	1/12/14	12:30– 1:15 P.M.	0.307	2:00 P.M.	0.068	0.039	930 000	45	0.22	0.12

* Ground dry at beginning of storm.

The Strawberry Creek records enabled the writer to determine run-off factors for a region of mild temperature and steep topography covered with grass and close growing brush. The percentage of water running off during the peak of the flood was found to be relatively small and the stream was frequently still swollen with delayed run-off from one period of maximum rainfall when another would occur. It was desirable to segregate the flow due to the current storm from that derived from previous storms. This was readily accomplished graphically upon the hydrographs by extending the slope of the diminishing flood flow to a point beneath the succeeding peak. The results for the ten storms are summarized in Table 10, which indicate that for major storms (January 12, 1914) occurring when the ground was still wet from previous storms, the ratio of maximum flood discharge to the rate of rainfall that produces it

* "Public Water Supplies," by Turneaure and Russell, Third Edition, p. 74.
† *Minutes of Proceedings*, Inst. C. E., Vol. CXXXIV, 1897–98, Pt. IV, p. 313.

was 0.12. If delayed run-off from previous storms was added, the ratio became greater and reached 0.22 in one instance (afternoon, January 12, 1914). If it be assumed that the run-off from the preceding storm is still at peak flow when the peak from the succeeding storm arrives, the run-off ratio (in terms of rainfall) for the record storm might be 0.24, which represents a limiting value. For maximum conditions, a value of 0.25 was assumed.

It is thus apparent that the flood discharge run-off factor has a wide variation, depending on local conditions. This element in the application of the rational method is the weakest link. The writer regards the method as having real merit and believes that attention should be given by the profession to perfecting its application.

OREN REED,* ASSOC. M. AM. SOC. C. E. (by letter).—A review of recent engineering literature would indicate that flood intensity and frequency have not been given their due importance. Widespread interest has been manifested after each major disaster, but more studies should be made for preventative purposes. The year 1927 was a remarkable period for great floods claiming the attention of the engineer. In them, he finds a need for thorough study as a means of coping with repetitions.

When floods occur, men instinctively feel that they are helpless against such occurrences, and that they must continue to bear them even in the future. It is recognized by engineers, of course, that there have always been floods and that their frequency and intensity have not changed greatly. Loss of property and life have steadily increased because Man has encroached on the river banks.† When structures are erected on the bank of a river, engineers must design with reference to flood discharges and flood stages. Their only safe criteria in estimating floods of the future must be obtained from a study of floods of the past. At present, unfortunately, the knowledge of flood facts is quite inadequate.

The record of flood run-off from any California river is too short to estimate accurately its probable maximum flood. Since 1918 the precipitation has been sub-normal, and no major floods have occurred. The passage of years will make the record more complete, but in the meantime existing records must be used.

Further Flood Study.—As stated by Mr. Fox, the writer's methods of studying flood records have been changed in certain details. In Central California, as in other regions, the geographic location of an area in respect to the path of cyclonic storms determines to a great extent its flood run-off. The primary data are used to determine only one curve, such as Fig. 7, which was called the "geographical location curve".

The relation of average elevations and size of drainage areas is, however, of major importance. Storms which produce the maximum flood run-off on the Kings River occur in January and February. At this season precipitation on the upper part of the water-shed is in the form of snow and contributes only a small run-off. Due to this fact the productive or effective area would be a

* Asst. Designing Engr., San Joaquin Light & Power Corporation, Fresno, Calif.

† *Engineering News-Record*, November 10, 1927, p. 741.

relatively small part of the total. This extended study has been based on effective areas.

Temperature data are not published for many mountain stations in California. Reliable records are available for the Fresno-Auberry-Huntington Lake Section, on the San Joaquin River, and for the Sacramento-Auburn-Colfax-Norden Section, on the American River. (See Fig. 18.) The American River was not used for the run-off records, but may be used in the study of temperature variation without error. The American River is about 35 miles north of the Cosumnes River, the most northerly stream in the run-off study.

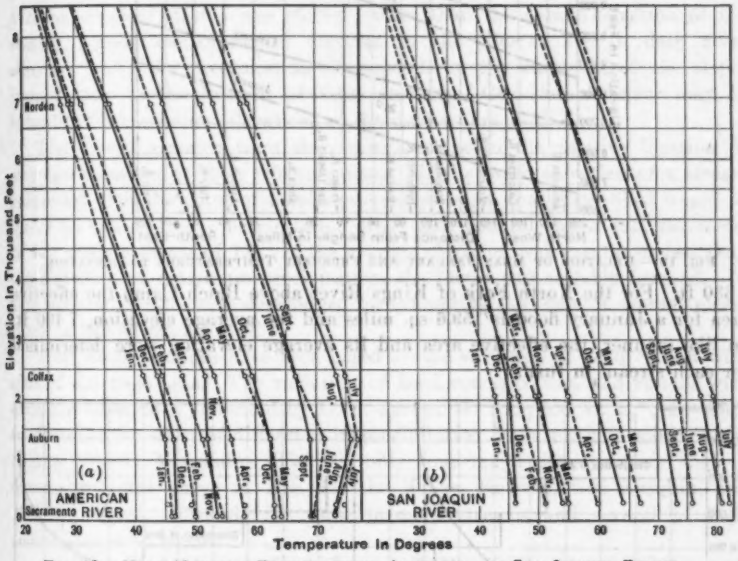

FIG. 18.—MEAN MONTHLY TEMPERATURES, AMERICAN AND SAN JOAQUIN RIVERS.

A study of meteorological data shows that the mean daily temperature during a storm period is from 4 to 8° higher than the mean monthly temperature. Based on that fact, the assumption has been made for this study that the run-off would be so slight that it could be neglected when the mean monthly temperature was below 22.5° Fahr. Fig. 19 has been drawn from the data given on Fig. 18 to show the relation of altitude to mean January and February temperature of 22.5° for the eleven streams. A curve for 25° is also given for reference. The curves are based on observations at only two stations and are plotted as straight lines. No doubt, there would be some variation from a straight line, but the deviation should not be great. This statement is substantiated by the temperature observations at the Upper Kings River meteorological stations. The Kings River stations have been maintained for six years.

The distribution of drainage area according to elevation was determined for each stream by planimeter and then plotted. Fig. 20 shows two of the

curves; one for the San Joaquin River at Friant (Fig. 20 (*a*)) and the other for the North Fork of Kings River above Balch Camp (Fig. 20(*b*)). Referring to Figs. 19 and 20(*a*), the effective area for the San Joaquin River, that would contribute to a January flood, was estimated to be 1 170 sq. miles, or 71.4% of the total area. The average elevation of the effective area was

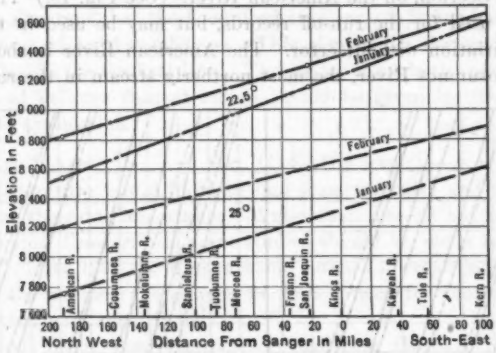

FIG. 19.—RELATION OF MEAN JANUARY AND FEBRUARY TEMPERATURES TO ELEVATION.

5 630 ft. For the North Fork of Kings River above Balch Camp the effective area for a January flood is 159.6 sq. miles and the average elevation, 7 490 ft. In like manner, the effective area and its average elevation were determined for each stream in turn.

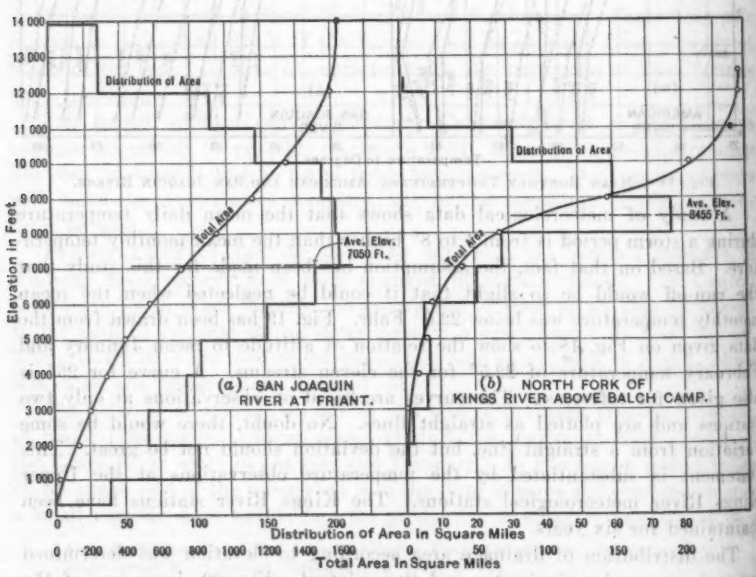

FIG. 20.—DISTRIBUTION OF DRAINAGE AREA.

It will be noted from Fig. 3, that the greatest precipitation from a normal storm on Kings River is about 6 000 ft. The locus of condensation on a mountain slope is usually at a lower level in winter than in summer, but this is offset to a certain extent by the low run-off of the upper part of the contributing area. Sufficient precipitation data were not available for each stream for plotting curves similar to Fig. 3 and only five curves were used, the Kern, Kings, San Joaquin, Mokelumne, and American Rivers. The San Joaquin River Section is indicated on Fig. 21. Each curve was used for two or three adjoining streams; the American River curve was used for the Cosumnes River only. In plotting these curves, the mean elevation of the slopes back of a meteorological station was plotted rather than the actual elevation of the station. Most meteorological stations in the Sierras are in deep river canyons while the precipitation corresponds to the elevation of the slopes immediately to the leeward of these stations. Considerable error may be introduced when actual elevations of the stations are used.

The eleven streams were then compared as to the relative location of average elevation of their effective areas. If the mean elevation of a stream coincided with the point of maximum precipitation, that water-shed would be likely to receive heavy precipitation from each general storm and, therefore, it was given a factor of 1.0. If the average elevation was greater or less than the elevation of maximum precipitation, the factor was suitably lowered. In this manner, factors were determined for evaluating the effect of average elevations of the eleven streams.

Most flood formulas have been based on the variation of flood run-off with size of drainage area. The variation of flood concentration and run-off with area has been ably presented by C. S. Jarvis, M. Am. Soc. C. E.* In order to decrease the effect of difference in precipitation from north to south in the Sierra Nevada Mountains (Fig. 2), only four major streams were used in determining the curve in Fig. 22—the Kings, Kaweah, San Joaquin, and Merced Rivers. Records for these primary streams and several of their tributaries were plotted.

Adjusted Probable Floods at Sanger, Calif.—Curves given in Figs. 7, 3, 21, and 22, and the variation factors for the streams given in Table 1 were used to determine the adjusted probable maximum flood on Kings River at Sanger, corresponding to the maximum 24-hour flood on each of the other streams. The San Joaquin River will be taken again as the example. The 24-hour maximum flood on the San Joaquin River was 38 800 sec-ft., or 33.16 sec-ft. per sq. mile of the effective area. This flood was 226% greater than the mean 24-hour flood during a period of 13 years on the San Joaquin, while the maximum variation on Kings River was 253% greater than the mean.

In addition:

From Fig. 7, Correction for "Distance from Sanger":

San Joaquin River = 25.6 sec-ft. per sq. mile
Kings River = 23.9 sec-ft. per sq. mile
Correction = 93.3%

* *Transactions*, Am. Soc. C. E., Vol. 89 (1926), p. 985.

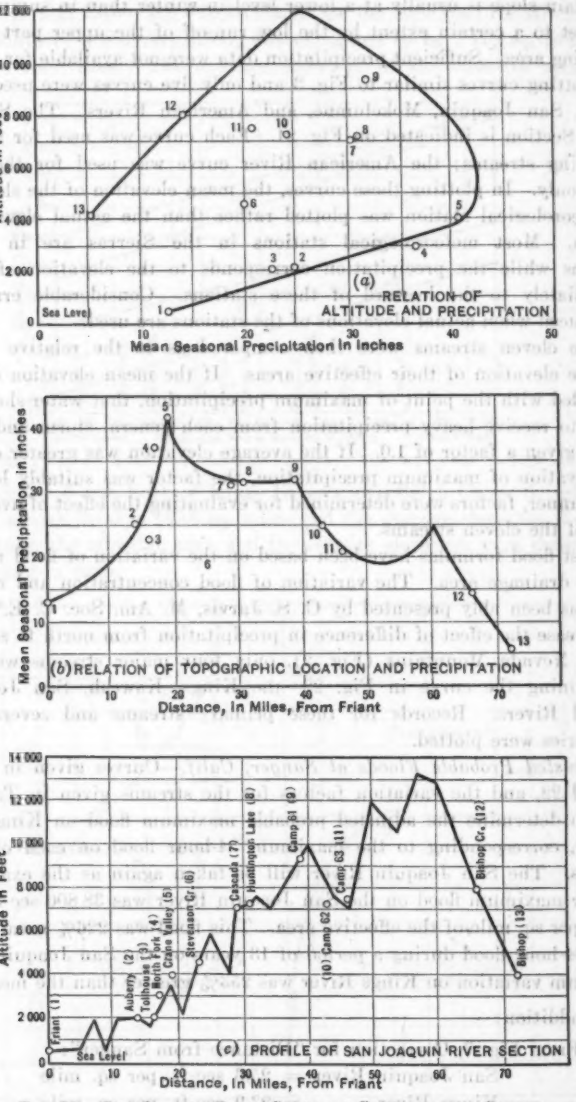

FIG. 21.—SAN JOAQUIN RIVER GROUP OF PRECIPITATION GAUGES.

From Figs. 21 and 3, Correction for "Average Elevation of Drainage Area":

San Joaquin River area above 9 100 ft. not productive:

Effective area = 1 169.4 sq. miles
Average elevation = 5 630 ft.

Kings River area above 9 200 ft. not productive:

Effective area = 1 191 sq. miles
Average elevation = 5 230 ft.

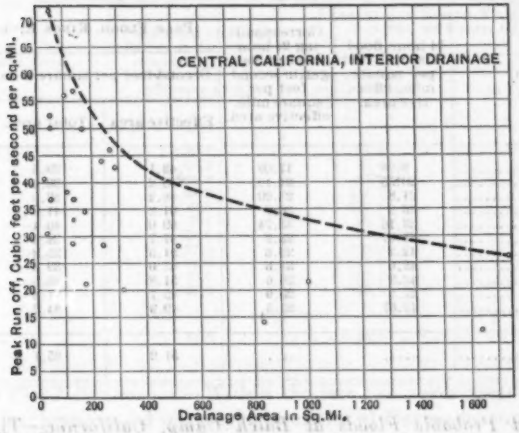

FIG. 22.—RELATION OF PEAK FLOOD TO SIZE OF DRAINAGE AREA.

San Joaquin River:

$$\text{Relative wetness} = \frac{43}{43} = 1.00$$

Kings River:

$$\text{Relative wetness} = \frac{46}{47.2} = 0.977$$

$$\text{Correction} = \frac{0.977}{1.00} = 97.7\%$$

From Fig. 22, Correction for "Drainage Area in Square Miles":

San Joaquin River... 1 169.4 sq. miles = 31.5 sec-ft. per sq. mile
Kings River..... 1 191 sq. miles = 31.4 sec-ft. per sq. mile
Correction..... = 99.7%

The maximum probable 24-hour flood at Sanger is,

$$33.16 \times \frac{253}{226} \times 93.3 \times 97.7 \times 99.7 = 33.74 \text{ sec-ft. per sq. mile}$$

or, 40 200 sec-ft. from the effective area.

From Fig. 10, a peak flood of 70 300 sec-ft., or 59.0 sec-ft. per sq. mile of contributing area, would be possible from a 24-hour flood of 40 200 sec-ft.

The other ten streams were studied in like manner and gave results as shown in Table 11.

The relation of the adjusted floods appears to be more reasonable than the values given in Table 3. The average value of the peak flood for Kings River, at Sanger, is only 2% greater than the peak flood of 1914. Peak flood intensity, as in 1914, is estimated from an extension of the rating curve and may be somewhat inaccurate.

TABLE 11.—DEDUCED FLOOD FLOW FOR KINGS RIVER NEAR SANGER, CALIFORNIA.

Stream.	24-hour flood, in second-feet per square mile, effective area.	Corresponding 24-hour flood at Sanger, in second-feet per square mile, effective area.	Peak Flood, Kings River, at Sanger.		
			Second-feet per square mile.		Second-feet.
			Effective area.	Total area.	
Kern	8.08	17.09	43.1	29.5	51 300
Tule	20.55	24.10	48.2	33.0	57 400
Kaweah	21.8	23.20	48.2	33.0	57 400
Kings	36.9	36.9	61.2	41.9	73 000
San Joaquin	33.16	33.74	59.0	40.4	70 300
Fresno	28.07	23.3	47.7	32.6	56 800
Merced	42.2	26.6	51.5	35.3	61 400
Tuolumne	42.6	31.6	57.0	39.0	67 800
Stanislaus	42.9	26.6	51.8	35.5	61 800
Mokelumne	27.5	20.9	45.7	31.3	54 500
Cosumnes	42.67	25.5	49.9	34.2	59 500
Average	.....	.....	51.2	35.1	61 000

Adjusted Probable Floods at Balch Camp, California.—The maximum peak flood on the North Fork at Balch Camp was determined by a similar method. The maximum 24-hour flood on the San Joaquin adjusted to suit Kings River conditions produced a peak flood of 59.0 sec-ft. per sq. mile of effective area at Sanger. The corresponding peak at Balch Camp was found to be:

$$59.0 \times 181.5 \times 100 = 107.1 \text{ sec-ft. per sq. mile of effective area}$$
$$= 17\,100 \text{ sec-ft.}$$

The results of the complete study are given in Table 12.

Effect of Storage Regulation.—Construction of a storage reservoir, Lake Wishon, is planned (1928) for a site near Cliff Camp, on the North Fork, at Elevation 6 550. When this dam is built, only that area between Lake Wishon and Balch Camp will contribute to a winter flood at Balch Camp, except in an extraordinary season. Usually, the storage reservoir would be drawn down by the beginning of winter and all of a winter flood could be stored. The drainage area between Lake Wishon and Balch Camp is 71.27 sq. miles and its average elevation is 6 450 ft. The peak flood at Balch Camp from this intermediate area, estimated by the method outlined for the total area, would be 7 990 sec-ft., or 112.2 sec-ft. per sq. mile of effective area.

However, in a very wet series of years, the reservoir might be practically full at the time of a late winter storm. Pondage above the spillway level

would reduce the flood peak, but the run-off from the drainage area above the storage dam would materially add to a flood at Balch Camp.

Flow for the Balch Tunnel is diverted from a small pond, which has a total capacity of 1 565 acre-ft. and a surface area of 37.75 acres at its maximum elevation. This pond will usually be kept full, and its effect on floods at Balch Camp will be negligible.

TABLE 12.—DEDUCED FLOOD FLOW FOR NORTH FORK OF KINGS RIVER AT BALCH CAMP, CALIFORNIA.

Flood adjusted from	Peak flood on Kings River, at Sanger, in second-feet per square mile of effective area.	PEAK FLOOD, NORTH FORK OF KINGS RIVER AT BALCH CAMP.	
		In second-feet per square mile of effective area.	In second-feet.
Kern	43.1	78.2	12 500
Tule	48.2	87.4	13 950
Kaweah	48.2	87.4	13 950
Kings River	61.2	111.0	17 700
San Joaquin	59.0	107.1	17 100
Fresno	47.7	86.5	13 800
Merced	51.5	93.5	14 920
Tuolumne	57.0	105.2	16 800
Stanislaus	51.8	94.2	15 030
Mokelumne	45.7	82.9	13 240
Cosumnes	49.9	90.8	14 500
Average	51.2	93.1	14 860

European Flood Experience.—Within the last few years great loss of life and property has been caused in various countries of Europe by storms of high intensity. In several cases the flood flow was greater than had been known possibly for centuries. In contrast to conditions in America where stream-flow records are limited to relatively short periods, most European countries have an unbroken record of river behavior for many centuries past. Recent experience there indicates that long-time contingencies must be reckoned with in providing protection against floods.

The following empirical data, Table 13, which are based on many years of past experience, are applicable to streams of Germany and Central Europe.*

TABLE 13.—PRECIPITATION AND RUN-OFF FOR GERMAN STREAMS.

Location.	Precipitation, in inches per year.	RUN-OFF, IN SECOND-FEET PER SQUARE MILE.	
		Average flow.	Flood flow.
Plains	23.6 to 27.6	0.53 to 0.68	4.08 to 6.80
Foot-hills	35.4 to 78.7	1.63 to 2.72	136.0 to 407.0
Mountains	70.8 to 86.6	2.04 to 4.76	54.3 to 136.0

Discussions.—Mr. Follansbee gives some very interesting precipitation data for stations near the Continental Divide in Colorado. The writer thor-

* Hütte, Bd. III, 24th Edition, p. 634.

oughly agrees that the topography to the windward side of the region being studied is a controlling factor in determining variation in precipitation. General storms which produce heavy precipitation on the west slope of the Sierra Nevada Mountains travel in a southeasterly course from the Pacific Ocean. Before reaching the mountain slopes the storms pass over the Central Valley for a distance of 150 to 250 miles. The valley floor is relatively flat and ranges in elevation from sea level to about 400 ft. at Bakersfield, in the south end of the area.

As R. E. Horton, M. Am. Soc. C. E., has pointed out,* the winds approaching the Continental Divide from the west and southwest must blow for hundreds of miles over the Rocky Mountain plateau, for which the dominant elevation is about 7 000 ft. These winds lose nearly all the moisture derived from the ocean, which they originally contained, in ascending to this plateau, and the precipitation which they yield thereafter is apparently derived mainly from moisture picked up en route from evaporation. The general plateau extends at a more or less uniform elevation of about 7 000 ft., nearly to the Continental Divide, where a rise of 4 000 to 6 000 ft. occurs within a distance of a comparatively few miles.

The temperature and relative humidity of these winds govern the amount of precipitation. The records indicate a moderate increase of precipitation with elevation up to a certain height, after which the increase is more rapid to a maximum close to Elevation 11 000. The rate of increase at Corona on the Divide is less than at lower elevations. Pike's Peak is situated to the east of the Continental Divide and receives less precipitation than the corresponding elevation on the west slope. After the winds pass over the Continental Divide they are relatively dry, and precipitation at Pike's Peak is slight. If the distance between mountain chains is sufficient, the relative humidity of the winds will be increased, due to moisture derived from evaporation, as stated, and the second range of mountains will give rise to a second belt of high precipitation. This is illustrated by the heavy precipitation on the slopes of the Wasatch and Uinta Mountains in Utah. A case similar to that at Pike's Peak is shown by Fig. 21. The secondary ridge near Crane Valley receives high precipitation and robs the remainder of the San Joaquin drainage area of a considerable portion of its normal precipitation.

Many engineers have questioned the theory that precipitation decreases above a certain elevation. The contention is made that the records of precipitation for high elevations are insufficient in number and accuracy to warrant the general conclusion. It is true that the stations are few in number and that the accuracy of some of the records could be greatly improved. However, each case studied showed the same characteristics. Some of the stations, especially on the American River Section, have been established for many years and are well maintained.

The percentage of precipitation appearing as run-off increases up to an elevation of approximately 10 000 ft. in the Sierra Nevada Mountains (see Fig. 23). This is due primarily to a decrease in water losses (evaporation,

* *Engineering News-Record*, February 28, 1924, p. 355.

transpiration, etc.) above the timber belt, which thins out above 7 000 ft. The relation of run-off to precipitation for German rivers is given in Table 14.* A similar relation, no doubt, holds for the area studied and a relatively high run-off per square mile would be expected from the higher drainage areas.

TABLE 14.—PERCENTAGE OF PRECIPITATION APPEARING AS RUN-OFF FOR GERMAN RIVERS.

Location.	Ratio of run-off to precipitation, in percentage.
Plains	25 to 30
Foot-hills	50 to 70
Mountains	85

The points covered by Mr. Fox in his discussion have been previously enlarged upon by the writer. One fact might be further emphasized; that is, until many more records of precipitation and run-off are obtained, reliable estimates of probable future floods on any particular stream are difficult to make unless all the comparable data are used.

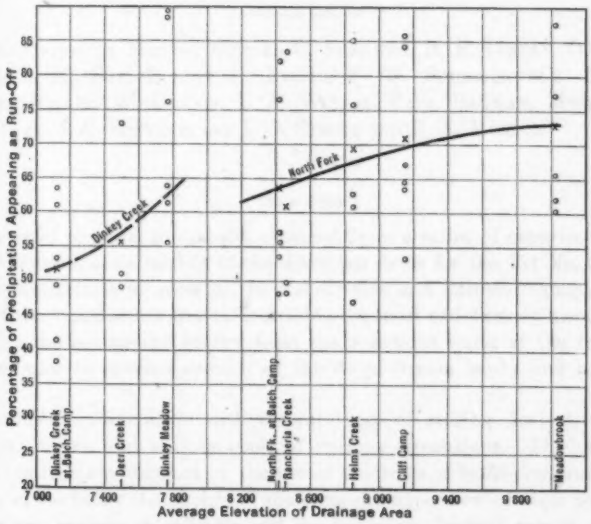

FIG. 23.—PERCENTAGE OF PRECIPITATION APPEARING AS RUN-OFF AT DIFFERENT ELEVATIONS, DINKEY CREEK AND NORTH FORK OF KINGS RIVER.

Mr. Lee's discussion is a valuable contribution to a study of flood discharge. When sufficient data are available, the rational method is the most reliable for studying flood behavior of a stream. Accurate records of precipitation intensity and amount must be collected for a considerable period for

* Hütte, Bd. III, 24th Edition, p. 634.

the area under consideration. The flood run-off factor can only be established by many observations. These data are largely lacking for drainage areas in the Sierra Nevada Mountains, and many years will be required to perfect the application of the rational method.

In the stream-group method, as used by the writer, the combination of intensity of rainfall and the percentage of resulting run-off make up the basic data. The record of maximum, 24-hour flood run-off for each primary stream, as determined by actual observation, was used in determining the equivalent record for the Kings River at Sanger.

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Paper No. 1706

BAFFLE-PIER EXPERIMENTS ON MODELS OF PIT RIVER DAMS*

By I. C. Steele† and R. A. Monroe,‡ Members, Am. Soc. C. E.

With Discussion by Messrs. Edward C. Sherman, B. E. Torpen, Oren Reed, Harold K. Fox, Robert F. Ewald, F. W. Scheidenhelm, Ralph L. Parshall, H. W. Dennis, I. A. Winter, Paul Bauman, Morrough P. O'Brien, J. C. Stevens, and I. C. Steele and R. A. Monroe.

Synopsis

This paper presents the results obtained from a series of experiments made on one-twentieth scale models of the diversion dams for the Pit No. 3 and Pit No. 4 hydro-electric projects of the Pacific Gas and Electric Company. The object of the experiments was to determine the most satisfactory means of controlling or destroying the energy from the over-pour water at the foot of the dams in order to prevent erosion of the down-stream banks and bed of the stream.

Experiments were made with several types of stilling devices, including baffle-piers, weirs, and stilling pools of various dimensions. The most satisfactory results were obtained by the use of two rows of baffle-piers resting on a concrete apron below the bucket of the dam, the upper row of piers being truncated prisms serving as splitters and the lower row having curved up-stream faces to act as deflectors and baffles.

Note.—The Special Committee on Irrigation Hydraulics selected the subject of "Scouring Below Dams" as one of ten for study and research. This paper was submitted to the Committee by its authors and the Committee has recommended its publication. (See Progress Report of the Committee, *Proceedings*, Am. Soc. C. E., March, 1927, Society Affairs, p. 124).

* Published in November, 1927, *Proceedings*.

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Pit No. 3 Dam

General Description.—Pit No. 3 diversion dam is a concrete structure of the ogee spillway type, 112 ft. high from foundation level to spill crest, with a spillway section, 18 ft. deep and 267 ft. long, designed to pass a flood of 70 000 sec-ft. The dam is arched in plan to a radius of 500 ft. Below the dam a concrete apron, with minimum thickness of 4 ft., supporting two lines of baffle-piers, extends a maximum distance of 160 ft. down stream from the toe of the dam. This apron is constructed at approximately river-bed level. (See Fig. 1.)

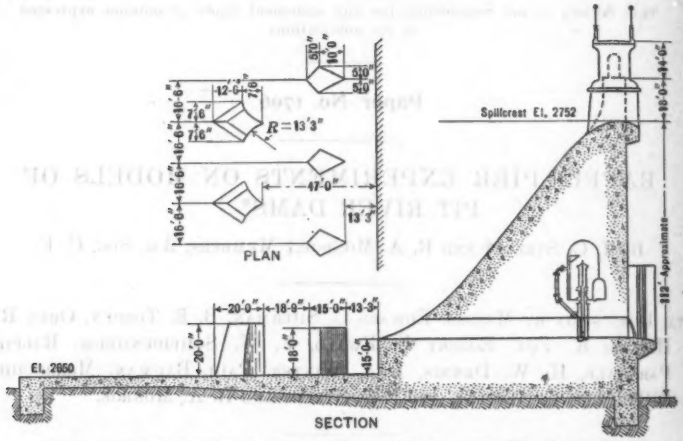


Fig. 1.—Arrangement of Baffle-Piers Adopted for Pit No. 3 Dam.

Unusual foundation conditions at the site of the dam influenced the adoption of concrete baffle-piers in conjunction with the lining of the river channel to guard against possible scouring and erosion at the toe of the dam. The prevailing rock formations in the Pit River Basin are either basaltic or andesitic lavas, lava agglomerate, or tuff, which vary from solid rocks capable of withstanding almost any load to soft friable masses. A thorough investigation of three miles of river channel by means of diamond-drill and drift work was made in order to determine the best possible site for the dam. Explorations at the site disclosed four alternating strata of basaltic lava rock and volcanic tuff lying in nearly horizontal position, but having a slight dip down stream. It was evident that the dam would be founded for the major part on the tuff with rock abutments. Tests of small laterally unsupported cubes of the tuff showed it to have an ultimate strength equivalent to from 50 to 75 tons per sq. ft. which was considered entirely safe as the foundation pressures under the dam are only about 8 tons per sq. ft. This material is firm, dense, and impervious, and very resistant to erosion, as was evidenced by the fact that although the river had cut through the upper rock formation, it had cut into the tuff only to a small extent.

It was felt, however, that during the alternate periods of wetness and dryness some weathering would take place and also that the tremendous

energy from the over-pour in times of flood might erode the tuff if left unprotected. It was not deemed advisable to attempt construction of any form of stilling pool because of the limited thickness of the tuff strata and because it was desired to leave intact as much of this impervious material as possible. Further, the banks immediately below the dam required protection, consequently it was decided to line the river channel with concrete for approximately 150 ft. down stream from the dam and to construct two rows of baffle-piers placed on, and incorporated with, the apron. The toe of the dam is designed with a 15-ft. vertical face, or jump-off, which extends the full width between the concrete wing-walls. As will be noted in Fig. 1, the tangent to the curve of the down-stream face of the dam at this jump-off is downward rather than horizontal. This was so designed to reduce the base width of the dam and to direct the spill water at this point so that the stream would strike well down on the up-stream or splitter piers and downward into any high back-water. If the tangent to the curve were horizontal at this point there would be a greater tendency for the water to jump over the piers, thus making them more or less ineffective at the very time they were most needed.

Tests on Model of Pit No. 3 Dam.—As there was little precedent for the use of baffle-piers, it was decided to construct a model of the dam and conduct a series of experiments to determine the proper shapes, sizes, and location of the piers. These experiments proved to be exceedingly interesting and enlightening and led to the selection of piers and layout considerably at variance with those first proposed.

Advantage was taken, in the design and construction of a small concrete diversion dam in Rock Creek, the waters of which are diverted into Pit No. 3 Tunnel, to carry out a series of experiments to determine a suitable system of baffles. This dam was designed and built as a one-twentieth scale model of the main structure in so far as features were concerned, which related to the problem at hand. A simple removable flash-board arrangement was installed to store water behind the dam in order to secure a maximum head equivalent to one-twentieth of that for which the main dam was designed.

While there is some question as to the action with any given depth of flow over such a model, as compared with that for the full-scale structure and its corresponding overflow, it is believed that the results obtained clearly indicated the relative efficiency of the several layouts and that the baffle arrangement as actually constructed will give excellent results.

During the early experiments an attempt was made to measure with a Pitot tube the velocity of the water flowing over the apron. No reliable information, however, was obtained owing principally to the strong eddy currents and the foamy condition of the water. An attempt also was made to take measurements of the trajectory of the stream below the apron in order to determine by calculation the velocity of the water as it left the apron. An inclined gauge was set up for this purpose, but it was found that, due to the commotion of the water, readings could not be taken with any degree of accuracy.

It was finally decided that close observation and notation of the action of the water during the various experiments and photographic study were the

best means of obtaining results. The experiments, therefore, were carried on in this manner and the notes and photographs carefully compared. Additional check experiments were then made to determine the plan that appeared to give the best results.

The experiments were made with various arrangements using three main groups of piers, as follows:

(1) Truncated pyramid-shaped piers.
(2) Prisms, or diamond-shaped piers.
(3) Combination of triangular prism piers and piers with curved up-stream faces approximating an impulse-wheel bucket in horizontal cross-section.

The spacing and types of piers, the gauge height or depth of overflow, the control height, and remarks relating to the observed results are given in Figs. 2, 3, and 4, and Tables 1, 2, and 3, for each of the eighteen experiments. Views of the action of the various piers are shown in Figs. 5 to 10, inclusive.

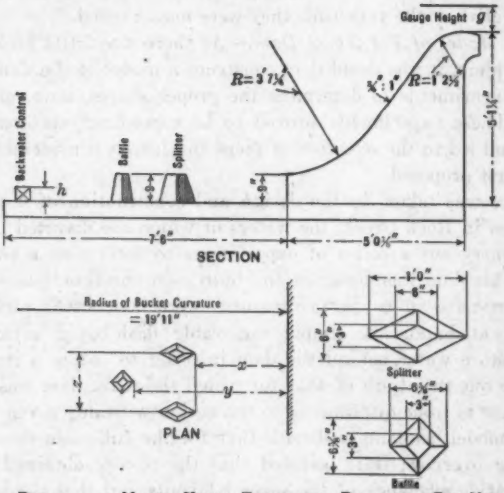


FIG. 2.—DETAILS OF MODEL USING TRUNCATED PYRAMIDS, PIT NO. 3 DAM.

All experiments, with the exception of Experiment No. 17, were made with two rows of piers set on arcs parallel to the curve of the front face of the dam and the bucket jump-off, the piers in the two rows being staggered. The up-stream piers were termed splitter-piers and those in the down-stream row, baffle-piers. Experiment No. 17 was made with only one row of curved baffle-piers.

The results may be summarized as follows:

1.—The first group of experiments using truncated pyramids gave very unsatisfactory results, the water leaping very high into the air as it left the splitter-piers.

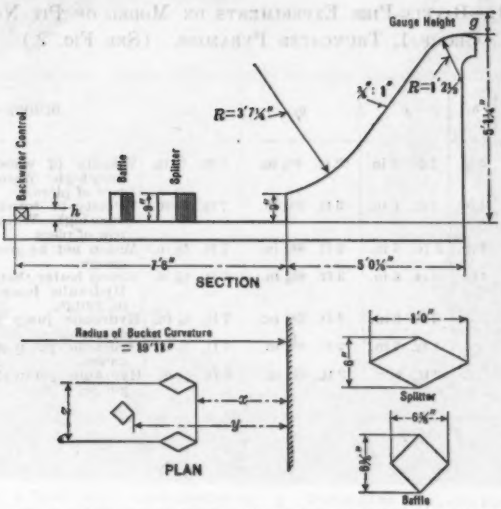


FIG. 3.—DETAILS OF MODEL USING PRISMS, PIT NO. 3 DAM.

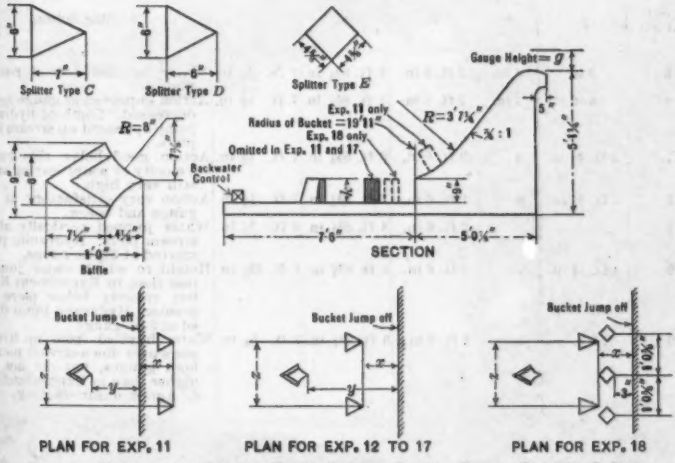


FIG. 4.—DETAILS OF MODELS USING CURVED BAFFLE-PIERS, PIT NO. 3 DAM.

TABLE 1.—BAFFLE-PIER EXPERIMENTS ON MODEL OF PIT NO. 3 DAM. GROUP 1, TRUNCATED PYRAMIDS. (SEE FIG. 2.)

Experiment No.	*g*.	*h*.	*x*.	*y*.	*z*.	Stilling action.
1	9 in.	0 in.	2 ft. 6 in.	3 ft. 8¾ in.	7 ft. ½ in.	Velocity of water leaving apron very high. Water hurdled second row of piers.
2	9 in.	4 in.	2 ft. 6 in.	3 ft. 8¾ in.	7 ft. ½ in.	Velocity of water leaving apron very high. Water hurdled second row of piers.
2	7 in.	4 in.	2 ft. 6 in.	3 ft. 8¾ in.	7 ft. ½ in.	Action not as good as with 9-in. gauge.
2	6 in.	4 in.	2 ft. 6 in.	3 ft. 8¾ in.	7 ft. ½ in.	Action better than at 7-in. gauge. Hydraulic jump started at 3½-in. gauge.
3	....	....	3 ft. 0 in.	4 ft. 2¾ in.	7 ft. ¼ in.	Hydraulic jump started at 3-in. gauge.
4	....	....	1 ft. 6 in.	2 ft. 8¾ in.	7 ft. ⅞ in.	Hydraulic jump started at 2¾-in. gauge.
5	....	....	2 ft. 6 in.	3 ft. 8¾ in.	9 ft. ⅝ in.	Hydraulic jump started at 3¼-in. gauge.

TABLE 2.—BAFFLE-PIER EXPERIMENTS ON MODEL OF PIT NO. 3 DAM. GROUP 2, PRISMS. (SEE FIG. 3.)

Experiment No.	*g*.	*h*.	*x*.	*y*.	*z*.	Stilling action.
6	9 in.	4 in.	2 ft. 6 in.	3 ft. 8¾ in.	7 ft. ½ in.	Water hurdled second row of piers.
6	6 in.	4 in.	2 ft. 6 in.	3 ft. 8¾ in.	7 ft. ½ in.	Action improved as gauge height decreased. Comb of hydraulic jump appeared up stream from piers.
7	5 ft. ¾ in.	0	2 ft. 6 in.	3 ft. 8¾ in.	7 ft. ½ in.	Action good below this gauge. Velocity of water leaving apron still very high.
7	3 ft. ¾ in.	0	2 ft. 6 in.	3 ft. 8¾ in.	7 ft. ½ in.	Action very satisfactory at this gauge and below.
8	.....	....	2 ft. 6 in.	3 ft. 8¾ in.	9 ft. ⅝ in.	Water jumped vertically at up-stream piers. Hydraulic jump started at 3¼-in. gauge.
9	4 ft. ½ in.	....	2 ft. 6 in.	3 ft. 8¾ in.	1 ft. 4¾ in.	Height to which water jumped less than in Experiment No. 8, but velocity below piers was greater. Hydraulic jump started at 3-in. gauge.
10	.....	....	1 ft. 0 in.	3 ft. 8¾ in.	9 ft. ⅝ in.	Water hurdled from up-stream piers over down-stream piers at high gauges, but did not rise higher than in Experiment No. 8. Action unsatisfactory.

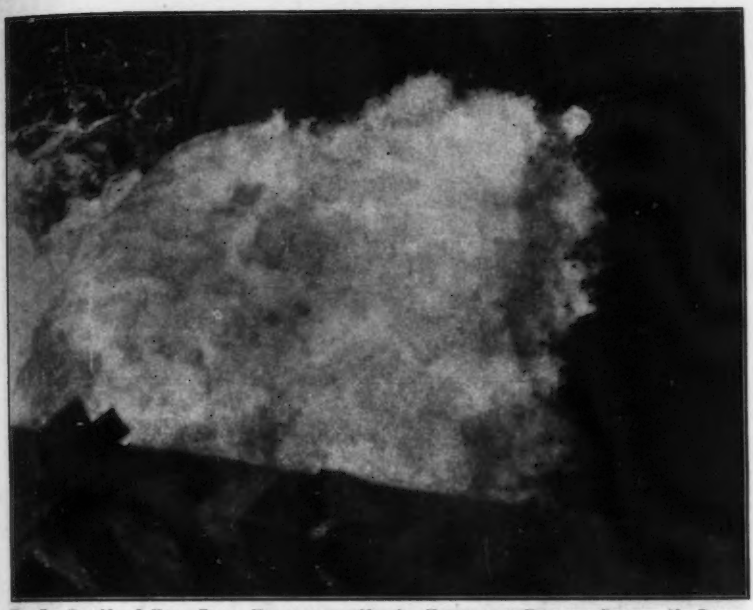
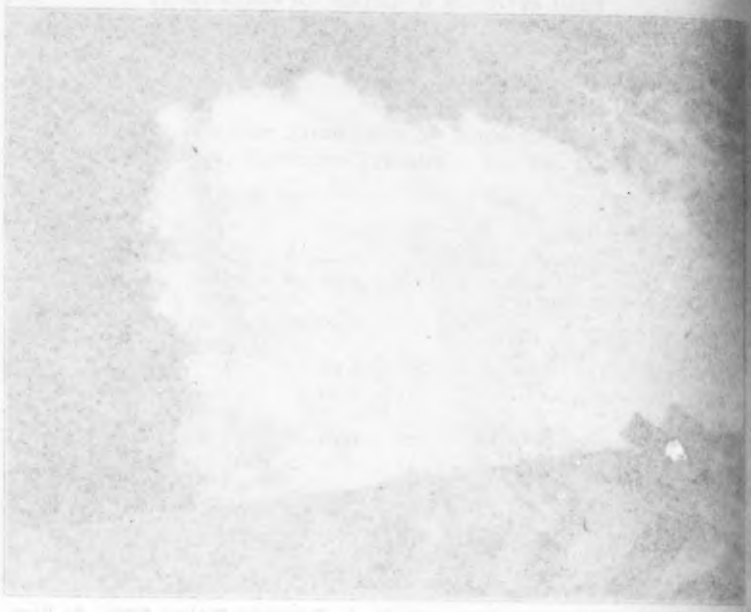


FIG. 5.—PIT NO. 3 TEST DAM: EXPERIMENT NO. 1. TRUNCATED PYRAMID PIERS. NO BACK-WATER CONTROL. GAUGE HEIGHT PROPORTIONAL TO 15 FEET ON ACTUAL DAM.

FIG. 6.—PIT NO. 3 TEST DAM: EXPERIMENT NO. 2. SAME AS FIG. 5, WITH BACK-WATER CONTROL PROPORTIONAL TO 6 FEET 8 INCHES ON ACTUAL DAM.



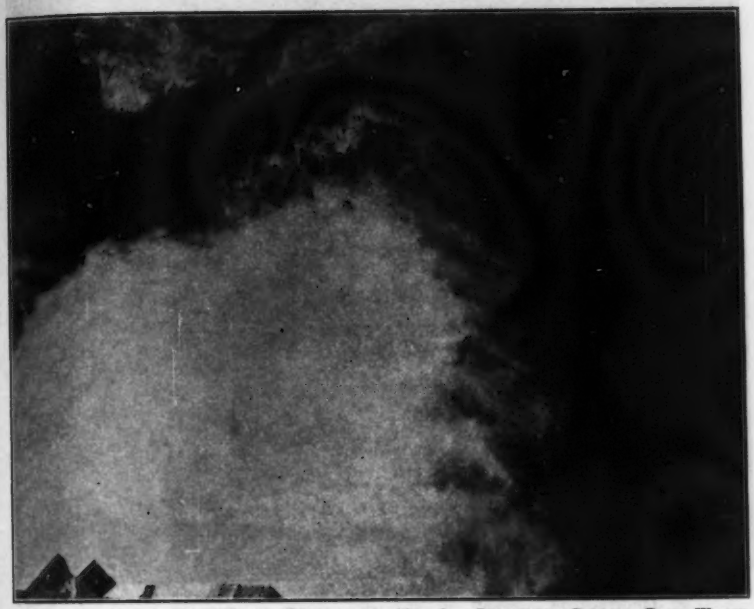


FIG. 7.—PIT NO. 3 TEST DAM: EXPERIMENT NO. 6. PRISMATIC PIERS. BACK-WATER CONTROL PROPORTIONAL TO 6 FEET 8 INCHES AND GAUGE HEIGHT TO 15 FEET ON ACTUAL DAM.

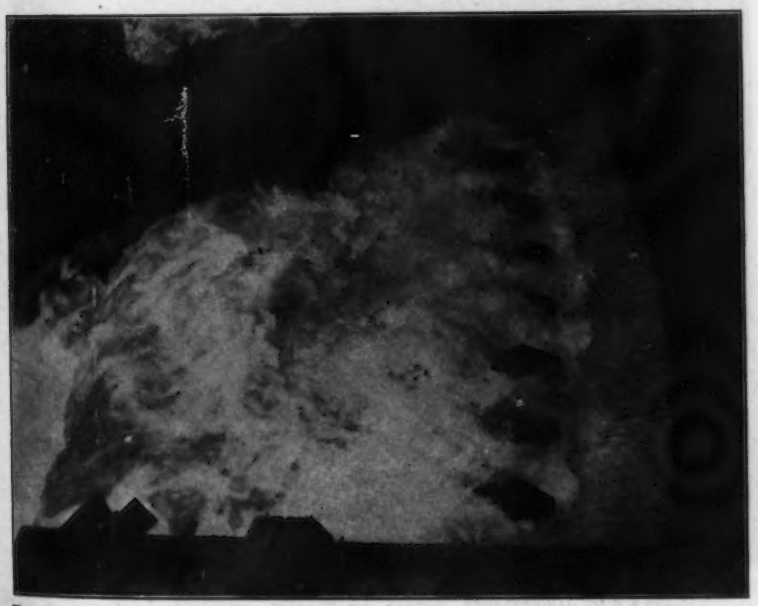
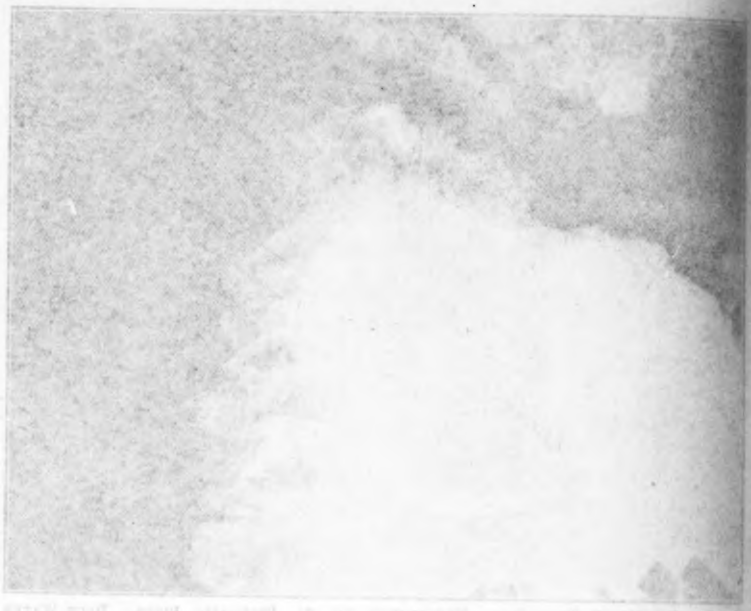


FIG. 8.—PIT NO. 3 TEST DAM: EXPERIMENT NO. 6. SAME AS FIG. 7, BUT WITH GAUGE HEIGHT PROPORTIONAL TO 10 FEET ON ACTUAL DAM.



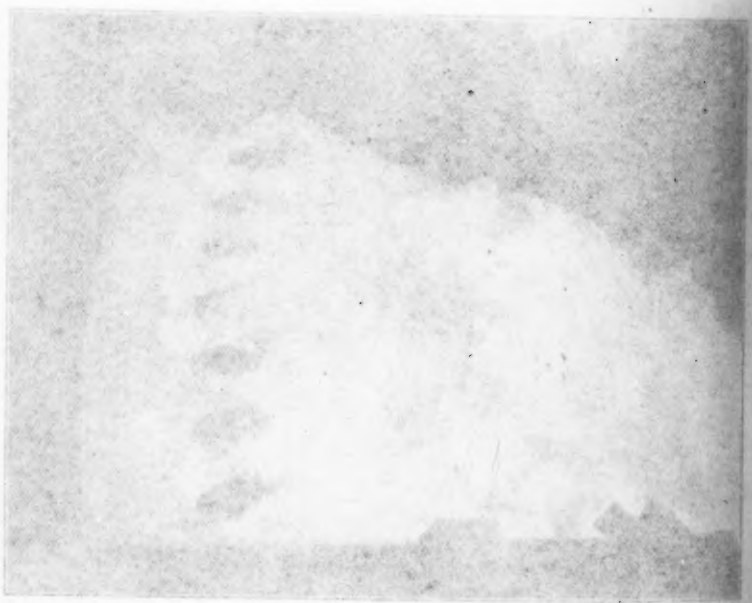
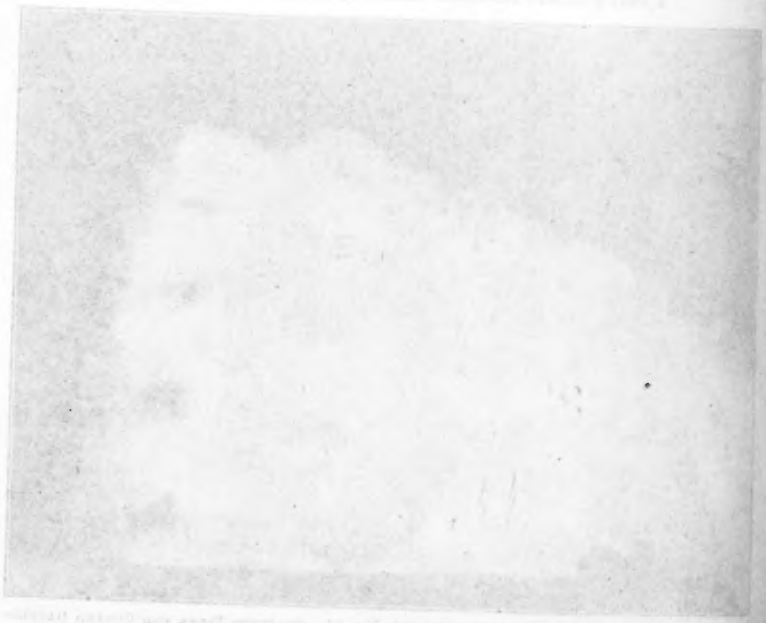


FIG. 9.—PIT NO. 3 TEST DAM: EXPERIMENT NO. 11. SPLITTER-PIERS ON TOE OF DAM. CURVED BAFFLE-PIERS ON APRON. BACK-WATER CONTROL PROPORTIONAL TO 6 FEET 8 INCHES AND GAUGE HEIGHT TO 15 FEET ON ACTUAL DAM.

FIG. 10.—PIT NO. 3 TEST DAM: EXPERIMENT NO. 13. SPLITTER-PIERS AND CURVED BAFFLE-PIERS ON APRON. BACK-WATER CONTROL PROPORTIONAL TO 6 FEET 8 INCHES AND GAUGE HEIGHT TO 10 FEET ON ACTUAL DAM.



2.—The second group of experiments using prism piers gave better results, but were still unsatisfactory.

3.—The third group of experiments using curved baffle-piers showed a marked improvement in stilling action. The best results were obtained by locating the splitter-piers close to the jump-off. Experiment No. 11, made to determine the effect of placing the splitter-piers on top of the jump-off, gave poor results. The water striking against these piers splashed very high on either side and in a manner very similar to the sidewash at the bow of a fast moving boat.

TABLE 3.—BAFFLE-PIER EXPERIMENTS ON MODEL OF PIT NO. 3 DAM. GROUP 3, CURVED BAFFLE-PIERS. (SEE FIG. 4.)

Experiment No.	*g.*	*h.*	*x.*	*y.*	*z.*	Type of splitter.	*k.*	Stilling action.
11	9 in.	4 in.	7 in.	1 ft. 0 in.	1 ft. 9 in.	D	9 in.	Results very unsatisfactory. Water splashed very high at splitter-piers.
11	6 in.	4 in.	7 in.	1 ft. 0 in.	1 ft. 9 in.	D	9 in.	Results very unsatisfactory.
12	9 in.	4 in.	8 in.	2 ft. 4 in.	1 ft. 9 in.	D	9 in.	Action better, but still unsatisfactory.
12	7 in.	4 in.	8 in.	2 ft. 4 in.	1 ft. 9 in.	D	9 in.	Action still better, but unsatisfactory.
12	5 in.	4 in.	8 in.	2 ft. 4 in.	1 ft. 9 in.	D	9 in.	Very good for gauges below this point.
13	9 in.	4 in.	8 in.	2 ft. 4 in.	1 ft. 9 in.	C	9 in.	Action excellent.
13	6 in.	4 in.	8 in.	2 ft. 4 in.	1 ft. 9 in.	C	9 in.	Action excellent.
13	4½ in.	4 in.	8 in.	2 ft. 4 in.	1 ft. 9 in.	C	9 in.	Action excellent.
14	.....	....	1 ft. 6 in.	2 ft. 6 in.	1 ft. 0½ in.	C	9 in.	At high gauges, water washed over tops of front piers and hurdled curved piers. Action good at lower stages.
15	.....	....	1 ft. 6 in.	2 ft. 6 in.	1 ft. 0½ in.	C	1 ft. 0 in.	Did not wash over piers, but jumped 3 ft. 9 in. vertically from them at gauge heights of 13 ft. 0 in., or more. Very good hydraulic jump occurred up stream from piers and between rows below 5½-in. gauges.
16	.....	....	1 ft. 6 in.	4 ft. 6 in.	1 ft. 1 in.	C	1 ft. 0 in.	Good. Curved piers caused good stilling action.
17	9 in.	4 in.	2 ft. 4 in.	.....	1 ft. 9 in.	....	.....	Action good. Did not compare with Experiment No. 13.
18	.....	....	1 ft. 0 in.	2 ft. 0 in.	1 ft. 9 in.	C and E	1 ft. 0 in.	Additional piers 3 in. from jump-off. Action good. Splashed to 2 ft. 3-in. height.

The layout finally adopted in the design and construction of the Pit No. 3 diversion dam gave excellent results. This was approximately that used in Experiment No. 13. The water flowing over the lower edge of the apron was comparatively quiet for all gauge heights and from all appearances no more harmful than the natural stream would be in time of high water.

Behavior of Completed Structure.—The Pit No. 3 Dam was completed in the summer of 1925 and, consequently, has passed through only two seasons of high flow. No floods were experienced in the spring of 1926 or 1927, the maximum flow of the river amounting to less than 8 000 sec-ft. The stilling action of the piers proved to be excellent. Continued observation during periods of high water will be maintained and eventually this structure will

afford valuable evidence of the success or failure of this method of preventing scour below dams.

Pit No. 4 Diversion Dam

General Description.—Pit No. 4 diversion dam is situated on Pit River approximately 9 miles down stream from Pit No. 3 diversion dam. On account of the peculiar foundation conditions at the site, the dam is designed as a composite structure. The foundation on the right bank and under the main channel of the stream is solid rock, whereas the left bank consists of a compact soil, gravel, and boulder formation. The design adopted for this site comprised a concrete ogee overflow spillway section from the right bank across the main stream channel, flanked by a flat-slab and buttress non-overflow structure on the left bank.

This dam is a relatively low structure, its function being to raise the water surface of the stream about 40 ft. The spill section of the dam is 136 ft. long, controlled by two drum-gates, each 68 ft. long by 14½ ft. high. Provision is made in the design for a maximum overflow depth of 27½ ft., which it is estimated will discharge a flood of 70 000 sec-ft.

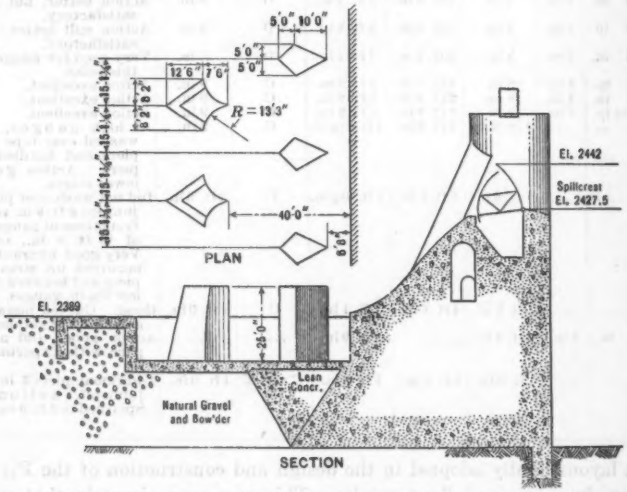


Fig. 11.—Arrangement of Baffle-Piers Adopted for Pit No. 4 Dam.

Below the dam a two-level concrete apron supporting two lines of baffle-piers extends a distance of 100 ft. down stream. The apron at the lower level has incorporated in its design a stilling pool, but this was incidental and was adopted as a measure of economy on account of the slope of the rock foundation and also to accommodate more conveniently the discharge openings through the dam. Fig. 11 shows a cross-section of the dam through the maximum or stilling-pool section and also indicates in plan the arrangement adopted for the baffle-piers.

Tests on Model of Pit No. 4 Dam.—The Rock Creek diversion dam which was constructed as a one-twentieth scale model of Pit No. 3 Dam, was reshaped with a wooden overflow and apron section to correspond to a one-twentieth scale model of the proposed Pit No. 4 Dam. A length of channel representing 100 ft. of river channel down stream from Pit No. 4 was also constructed to the shape of the actual river with loose rocks on a false wooden bottom. The general arrangement of the test model is shown by Fig. 19.

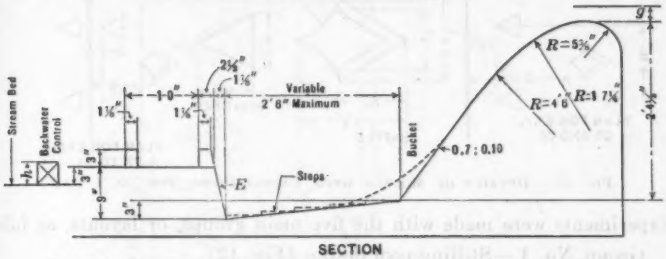


FIG. 12.—DETAILS OF MODELS WITH STILLING POOL, PIT NO. 4 DAM.

Various test runs were made by placing flash-boards across the overflow section of the dam and allowing the water in the reservoir above to rise to a gauge height corresponding to 28 ft. on the proposed Pit No. 4 Dam. This

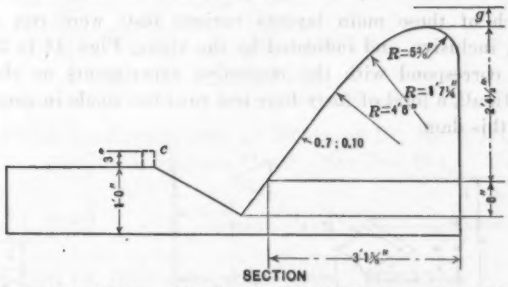


FIG. 13.—DETAILS OF MODEL WITH FRENCH DESIGN SETTLING POOL, PIT NO. 4 DAM.

allowed the overflow water, when the flash-boards were lifted, to come to a steady condition of flow at 27½ ft. representing the maximum expected flood.

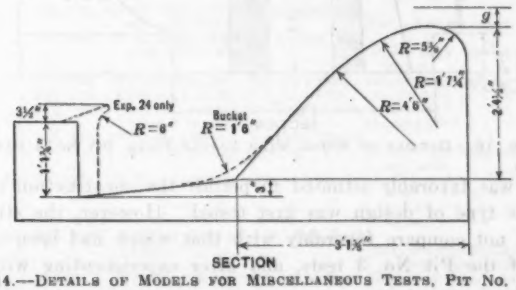


FIG. 14.—DETAILS OF MODELS FOR MISCELLANEOUS TESTS, PIT NO. 4 DAM.

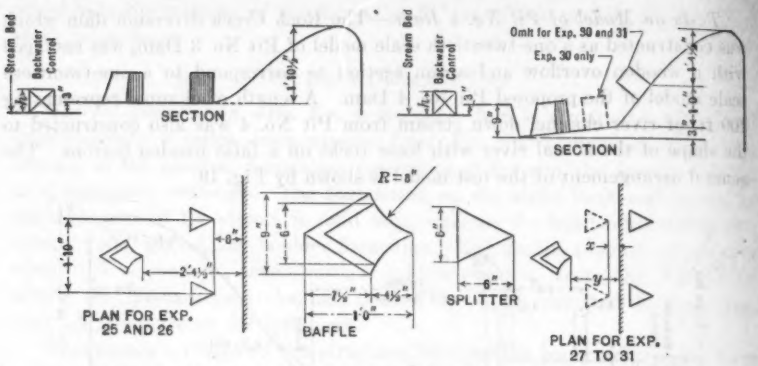


FIG. 15.—DETAILS OF MODELS WITH BAFFLE-PIERS, PIT NO. 4 DAM.

Experiments were made with the five main groups, or layouts, as follows:

Group No. 1.—Stilling-pool design (Fig. 12).
Group No. 2.—The French stilling pool with upward sloping bottom (Fig. 13).
Group No. 3.—Miscellaneous layouts (Fig. 14).
Group No. 4.—Baffle-pier designs (Figs. 15 and 16).
Group No. 5.—Baffle-pier designs, with two-level apron (Fig. 17).

Under each of these main layouts various tests were run as given in Tables 4 to 9, inclusive, and indicated by the views, Figs. 18 to 27, inclusive, numbered to correspond with the respective experiments as shown on the photographs. In all, a total of forty-four test runs was made in connection with the design of this dam.

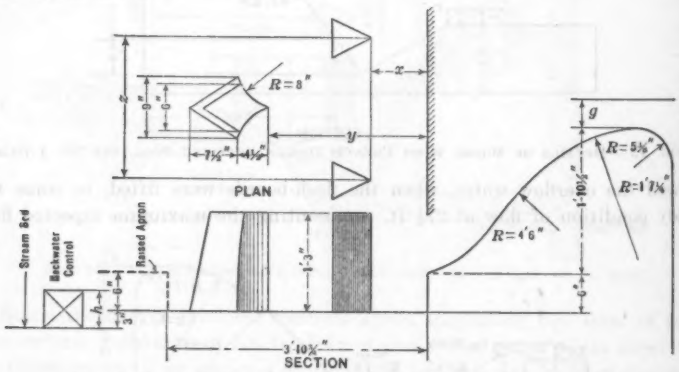


FIG. 16.—DETAILS OF MODEL WITH BAFFLE-PIERS, PIT NO. 4 DAM.

The site was favorably situated to permit the construction of a stilling pool and this type of design was first tested. However, the stilling action observed did not compare favorably with that which had been observed for baffle-piers of the Pit No. 3 tests, and after experimenting with the plain

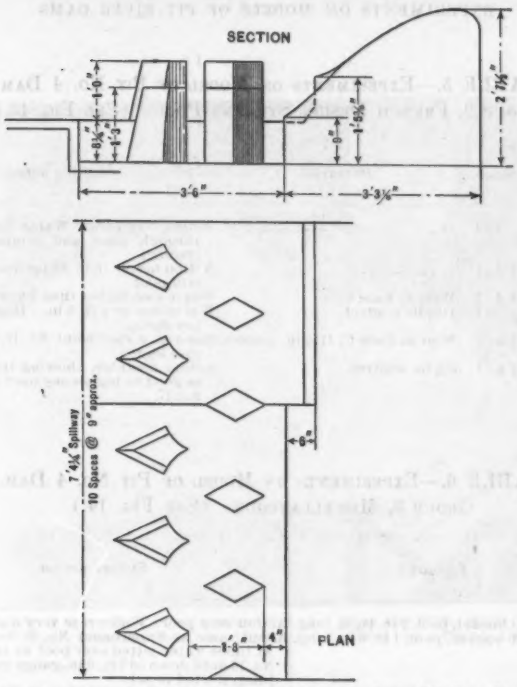


FIG. 17.—EXPERIMENTS ON MODEL OF PIT NO. 4 DAM, GROUP 5, TWO-LEVEL APRONS.

TABLE 4.—EXPERIMENTS ON MODEL OF PIT NO. 4 DAM.

GROUP 1, STILLING POOL. (SEE FIG. 12.)

Experiment No.	h.	Length of pool.	Remarks.	Stilling action.
1	.....	2 ft. 6 in.	With bucket, without weirs.	Action poor.
2	.....	2 ft. 6 in.	With bucket, without weirs, g = 11⅜ in.	Action poor.
3	.....	2 ft. 6 in.	With bucket and with both weirs.	Action poor. Water rose higher than without weirs.
4	.....	2 ft. 6 in.	Without bucket and with both weirs.	Action practically same as Experiment No. 3.
5	.....	2 ft. 6 in.	With steps, without bucket, and without weirs.	Steps had no effect.
6	.....	2 ft. 6 in.	With steps, without bucket, and with both weirs	Weirs only served to raise water higher.
7	.....	2 ft. 3 in.	Without bucket, without weirs.	Shortening pool had no effect.
8	.....	2 ft. 0 in.	Without bucket, without weirs.	Shortening pool had no effect.
9	.....	2 ft. 0 in.	Without bucket, without weirs. Face K vertical.	Action poor.
10	.....	2 ft. 0 in.	Without bucket. Vertical face of weir at vertical face, E.	Water rose even higher than without weir.
11	11⅜ in.	2 ft. 0 in.	Without bucket, without weirs.	Action poor. Water rose to 2 ft. 0 in. Control did not back water up to pool on account of high velocity.
12	11⅜ in.	2 ft. 0 in.	Without bucket and with both weirs.	Action same as Experiment No. 11, except water rose to 2 ft. 3 in.
13	.....	9 in.	Without bucket, without weirs.	Water washed over pool until gauge was down to 1 ft. 0 in. when hydraulic jump started.

TABLE 5.—EXPERIMENTS ON MODEL OF PIT NO. 4 DAM.
GROUP 2, FRENCH DESIGN STILLING POOL. (SEE FIG. 13.)

Experiment No.	Slope.	Remarks.	Stilling action.
14	2 : 1	.....	Action very poor. Water jetted smoothly through pool and jumped into river channel.
15	1.5 : 1	.....	Action better than Experiment No. 14, but still poor.
16	1.5 : 1	Weir at Face *C*.	Water rose higher than Experiment No. 15.
17	1.5 : 1	11⅛-in. control.	Water rose to 1 ft. 9 in. High velocity below apron.
18	1.5 : 1	Weir at Face *C*; 11⅛-in. control.	Same as Experiment No. 17, except water rose higher.
19	1.5 : 1	5¼-in. control.	Action unstable, showing this control not as good as higher one used in Experiment No. 17.

TABLE 6.—EXPERIMENTS ON MODEL OF PIT NO. 4 DAM.
GROUP 3, MISCELLANEOUS. (SEE FIG. 14.)

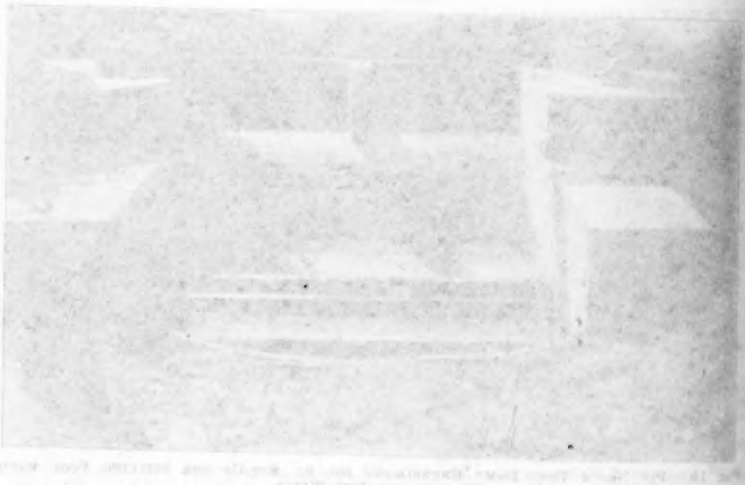
Experiment No.	Remarks.	Stilling action.
20	Without bucket, pool, 2 ft. 4⅜ in. long.	Action very poor. Water rose very high.
21	Without bucket, pool, 1 ft. 3 in. long.	About same as Experiment No. 20, but less stable. At times water jetted over pool as in Experiment No. 18 until down to 1 ft. 0 in. gauge when hydraulic jump started to act.
22	With bucket, pool, 2 ft. 7¼ in. long.	Action only slightly better than in Experiment No. 23 below.
23	With bucket, pool, 2 ft. 0 in. long.	Action not good. Very stable rising of water at up-stream edge of raised apron.
24	Without bucket, pool, 2 ft. 0 in. long. Curved up-stream face of apron.	Action very bad. Water rose 2 ft. 3 in. to 2 ft. 6 in. and washed down stream of apron.

TABLE 7.—EXPERIMENTS ON MODEL OF PIT NO. 4 DAM.
GROUP 4, BAFFLE-PIERS. (SEE FIG. 15.)

Experiment No.	*h*.	*x*.	*y*.	Remarks.	Stilling action.
25	.....	8 in.	2 ft. 4¼ in.	.....	Action poor. Water splashed to 2 ft. 6 in. height and down into river channel.
26	6⅞ in.	8 in.	2 ft. 4¼ in.	.....	Action same as in Experiment No. 25.
27	11⅛ in.	....	9 in.	Splitter on dam.	Action fairly good. Water rose 1 ft. 6 in. to 1 ft. 8 in. Slightly greater velocity in river channel than with Experiment No. 33 to 39.
28	5¼ in.	....	9 in.	Splitter on dam.	Action practically same as in Experiment No. 27.
29	11⅛ in.	....	1 ft. 1¼ in.	Splitter on dam.	Action not as good as in Experiment No. 27.
30	11⅛ in.	8 in.	1 ft. 7¼ in.	Splitter in pool.	Action poor. Water rose to 2 ft. 0 in.
31	11⅛ in.	....	1 ft. 7¼ in.	No splitter.	Poor stilling action. Water rose to 2 ft. 0 in. and had considerable velocity in river channel.

FIG. 18.—PIT NO. 4 TEST DAM: EXPERIMENT NO. 1. STILLING POOL WITH BUCKET. GAUGE HEIGHT PROPORTIONAL TO 27 FEET 6 INCHES ON ACTUAL DAM.

FIG. 19.—PIT NO. 4 TEST DAM: EXPERIMENT NO. 3. SET-UP FOR STILLING POOL WITH BUCKET AND TWO WEIRS.



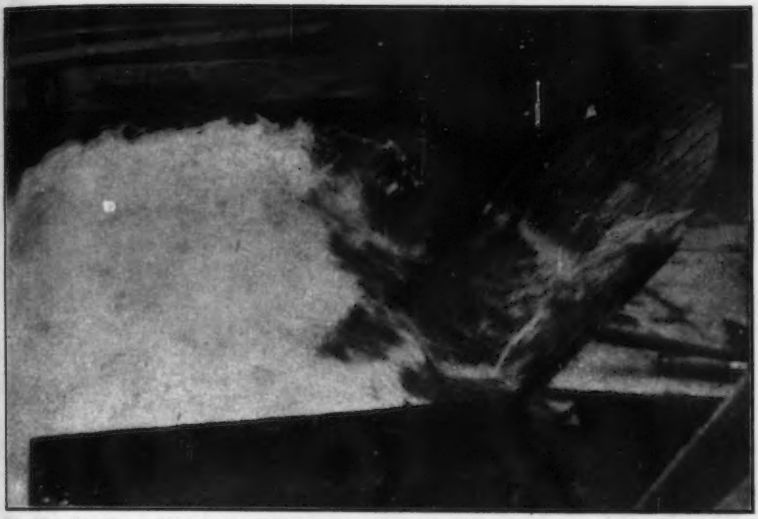


FIG. 20.—PIT NO. 4 TEST DAM: EXPERIMENT NO. 15. FRENCH DESIGN. 1.5:1 POOL SLOPE.

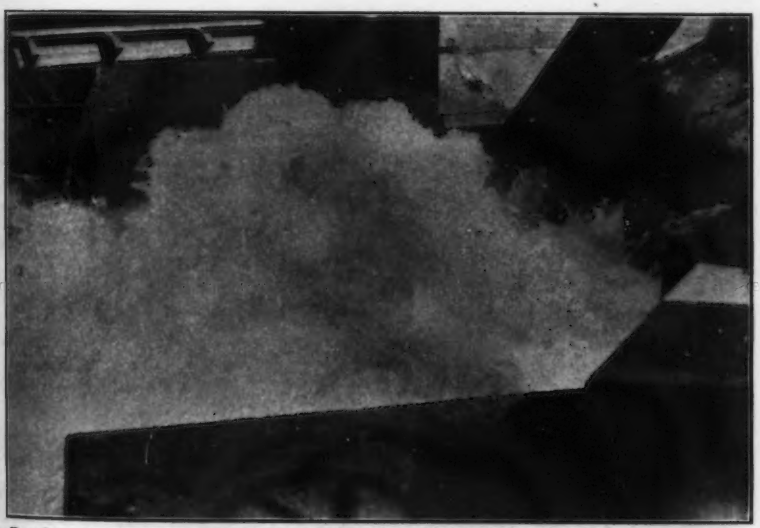


FIG. 21.—PIT NO. 4 TEST DAM: EXPERIMENT NO. 16. SAME AS FIG. 20, WITH ONE WEIR AT TOP OF SLOPE.

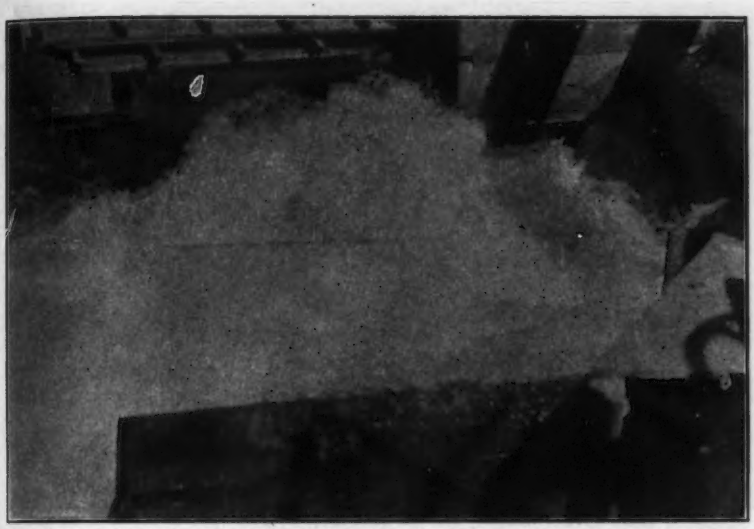


FIG. 22.—PIT NO. 4 TEST DAM: EXPERIMENT NO. 20. MISCELLANEOUS TYPES. NO BUCKET ON DAM. VERTICAL FACE AT APRON.

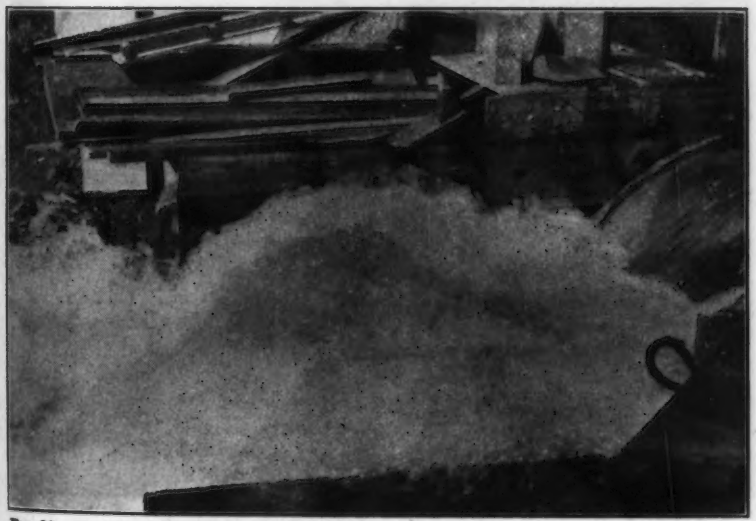


FIG. 23.—PIT NO. 4 TEST DAM: EXPERIMENT NO. 24. MISCELLANEOUS TYPES. NO BUCKET ON DAM. CURVED UP-STREAM FACE OF APRON.

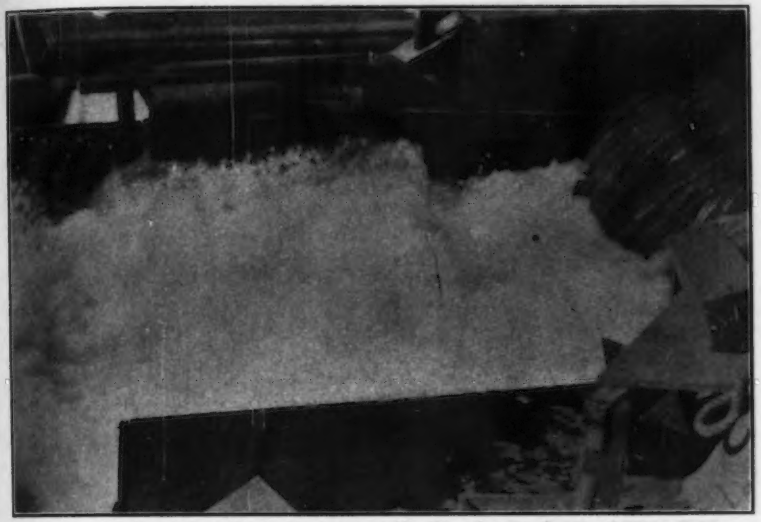


FIG. 24.—PIT NO. 4 TEST DAM: EXPERIMENT NO. 31. CURVED BAFFLE-PIERS WITHOUT SPLITTER-PIERS. BACK-WATER CONTROL PROPORTIONAL TO 19 FEET ON ACTUAL DAM.

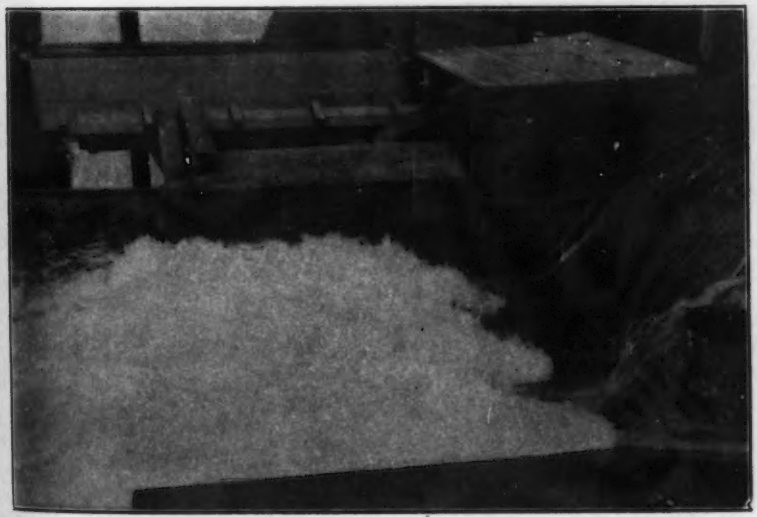


FIG. 25.—PIT NO. 4 TEST DAM: EXPERIMENT NO. 32. CURVED BAFFLE-PIERS WITH SPLITTER-PIERS AND CURVED BAFFLE-PIERS. BACK-WATER CONTROL PROPORTIONAL TO 11 FEET 6 INCHES ON ACTUAL DAM.

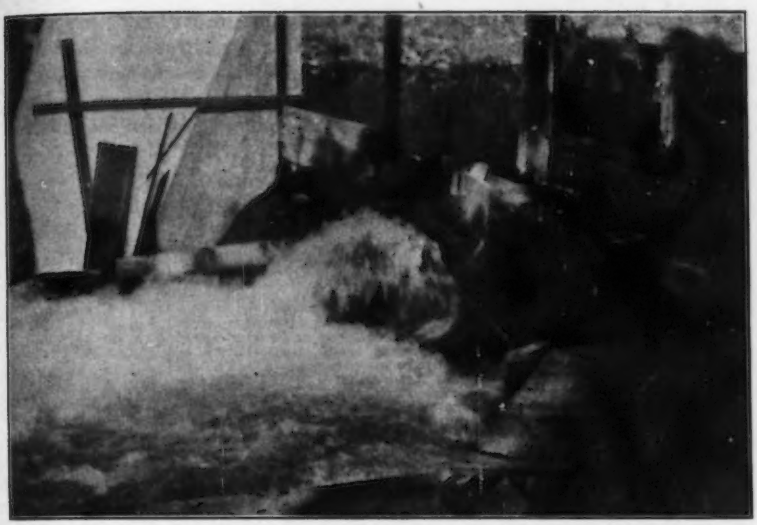


FIG. 26.—PIT NO. 4 TEST DAM: EXPERIMENT NO. 38. TWO-LEVEL APRONS WITH BAFFLE-PIERS.

FIG. 27.—PIT NO. 4 TEST DAM: EXPERIMENT NO. 40. SAME AS FIG. 26, EXCEPT RIGHT SPLITTER-PIER REMOVED.

stilling pool, the French stilling pool, and some miscellaneous layouts, it was decided to experiment with baffle-piers. The piers were duplicates of those at Pit No. 3 and spaced similarly. Splitter-piers, that is, the up-stream piers, were half spaced from the wing-walls, and the end pier in the line of curved piers was one-half the regular pier fastened against the wing-wall on one side.

TABLE 8.—EXPERIMENTS ON MODEL OF PIT NO. 4 DAM.
GROUP 4, BAFFLE-PIERS (*Continued*). (SEE FIG. 16.)

Experiment No.	*h*.	*x*.	*y*.	Remarks.	Stilling action.
32	6⅞ in.	8 in.	2 ft. 4¼ in.	.....	Stilling action very good.
33	6⅞ in.	8 in.	2 ft. 4¼ in.	$g = 1$ ft. 0 in.	Stilling action very good.
34	.....	4 in.	2 ft. 0¼ in.	River channel extended up stream to within 2 in. of baffle-piers.	Action practically same as Experiment No. 32.
35	9 in.	4 in.	2 ft. 0¼ in.	River channel extended up stream to within 2 in. of baffle-piers.	Action very good. Water below apron quiet.
36	5¼ in.	4 in.	2 ft. 0¼ in.	Raised apron.	Action very good. Water rose from 1 ft. 3 in. to 1 ft. 6 in.
37	11⅜ in.	4 in.	2 ft. 0¼ in.	Raised apron.	Action very good.

TABLE 9.—EXPERIMENTS ON MODEL OF PIT NO. 4 DAM.
GROUP 5, TWO-LEVEL APRONS. (SEE FIG. 17.)

Experiment No.	*h*.	Remarks.	Stilling action.
38	11⅜ in.	.....	Action good. Water thrown up by flat face of right splitter.
39	11⅜ in.	Right splitter reduced to half width.	Same as Experiment No. 38, except right splitter only threw thin sheet of water.
40	11⅜ in.	Right splitter removed.	Action very good except for water which washes up right wing-wall at bucket, as in all tests of this group.
41	11⅜ in.	Right splitter cut to fit against wing-wall.	Action same, except slightly more splash on right side.
42	5 in.	Right splitter as in Experiment No. 41. Splitter at step between apron levels increased to height of piers on high portions.	Reduced control did not effect action. Raising center pier caused better action.
43	5 in.	Same as Experiment No. 42, except piers spaced at 2 ft. 1¾ in.	Action not as good as previous experiments.
44	11⅜ in.	Same as Experiment No. 42.	Action better. Not as good as with closer pier spacing.

The experiments showed that the baffle-piers were by far the most effective destroyers of energy for such a deep over-pour as that proposed for Pit No. 4 Dam. In fact, none of the other layouts was satisfactory.

Of the arrangements of piers, the best was with a jump-off from the bucket of the dam, thus making the jet of water strike the piers at about mid-height With no jump-off, that is, with the overflow jet striking at the base of the piers, the water passes into the air in vertical sheets, unless there is considerable back-water from the channel below. With the jump-off the back-water, or lack of back-water, has little affect on the baffle-piers.

After a series of tests on a one-level apron a second series was undertaken with a two-level apron similar to that actually used in the design of the Pit No. 4 Dam. The model was accordingly altered to represent the two-level apron type. Results of tests of this layout and with slight modifications are shown in Fig. 11. The stilling action was not affected by the two-level feature. The action was just as satisfactory on the high apron side as if the whole apron was at that elevation. The "killing" of velocity was accomplished with less commotion on the deep side than on the shallow, but the difference in the final result, as indicated by the velocity in the river channel below, was, as nearly as could be judged, uniform across the width.

The flat up-stream face of the splitter-pier near the right wing-wall seemed to throw considerable water into the air. Even with a sharp cutting-edge, this pier seemed to do more harm than good.

Incidentally, it was noted that the edge of the over-pour water at the right end of the dam maintained a vertical face until reaching the bucket curve where it flattened out with a resulting component of velocity along the length of the dam. This water impinged against the sloping right-bank wing-wall and washed up this wall to a height corresponding to 40 ft. on the actual structure. It is thought that this action could be eliminated either by a training wall on the down-stream face of the dam, or by moving the wing-wall out so as to be in line with the end of the spill section of the dam.

Observations on Completed Structure—Pit No. 4 Dam was completed after the high-water season of 1927. It will not be possible to observe the action of the piers in destroying the energy of the over-pour water under actual flood conditions until the spring of 1928.

DISCUSSION

EDWARD C. SHERMAN,* M. AM. SOC. C. E. (by letter).—It is fortunate that such a complete series of experiments was possible on the models of Pit River dams. The writer concurs in the opinion that there is a question as to the flow over a model being entirely comparable with the corresponding flow over the full-sized structure. However, there can be no doubt that useful information may be obtained from such investigations.

From 1909 to 1911, the writer was Designing Engineer on the Panama Canal, and had charge of the preparation of plans for the spillway dams at Gatun and Miraflores. The larger is at Gatun, and for that location, consideration was given to two types: One with a straight crest and stepped section; the other curved in plan and of ogee section. Models of both were made, primarily to aid in determining the better type to use, but incidentally the opportunity afforded was utilized to test the action of baffle-piers in checking the velocity at the toe of the dam.

The lake formed by the great earth dam at Gatun, with its surface 87 ft. above sea level, has an area of 168 sq. miles, and receives the drainage from a water-shed of 1 320 sq. miles. The heavy precipitation of the rainy season frequently causes a high rate of run-off, and as records had been kept for many years, it was possible to determine that the probable maximum momentary rate of discharge at Gatun would be 183 000 sec-ft. The area of the lake is so great, however, that spillway capacity of this amount is not necessary, and the design was made to provide a discharge of 140 000 sec-ft. with the lake at its normal level of 85 ft. About midway in the length of Gatun Dam, there was a rock outcrop which afforded a suitable foundation for the spillway. The extent of this foundation determined the possible length of the crest and, consequently, its elevation. The total distance between abutments is 760 ft., giving a clear crest length of 630 ft., which is divided into fourteen 45-ft. openings. The canal locks were designed on the assumption that the summit level in Gatun Lake would not rise above Elevation 87, and to secure the required capacity without allowing the lake to rise higher, the spillway crest was placed at Elevation 69, with gates of the Stoney type for control.

Tests made with the model of the straight crest dam (Fig. 29) indicated that, with all gates open, the stepped section was effective in destroying the energy of the fall and that uniform flow could be expected below the standing wave at the toe of the dam if the down-stream edge of each step were provided with a strip which would form a pocket or pool. Gatun Lake is frequently regulated by opening a few gates, or only one. It rarely requires full spillway capacity. A test of the model, with only one gate open, gave unexpected results. It was noted that the stream, after striking the first step, turned upward to approximately lake level, without apparent loss of head, and shot forward with sufficient velocity to miss the other steps; so that most of the water fell directly on the floor at the toe of the dam without any check to its force. Similar action on a dam of full size would quickly

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destroy it; therefore, the results of the test influenced the decision to adopt a dam of ogee section, the model of which is shown in Fig. 30.

The crest of the dam is of a form which facilitates the flow while providing seats for the Stoney gates and the caisson required to permit repairs. The upper part of the down-stream face is parabolic, and shaped so that the nappe adheres to the masonry, and the lower part of the face is a circular curve of long radius. The general plan and sections of the spillway dam are shown in Fig. 28.

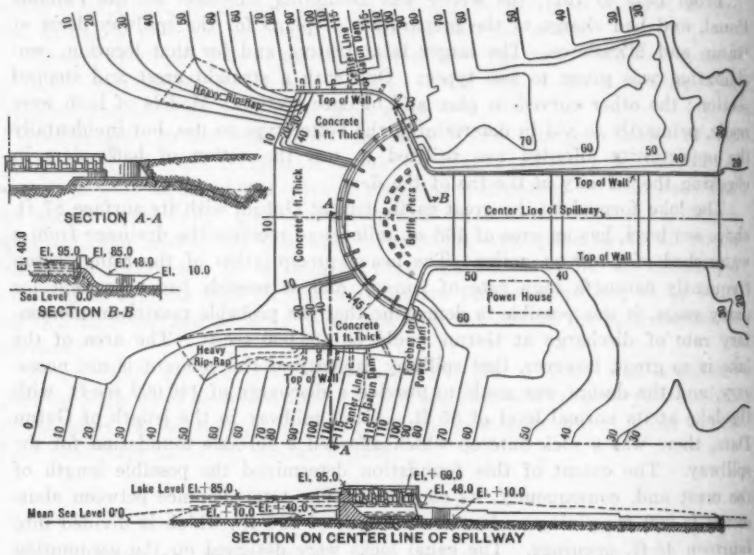


FIG. 28.—GENERAL PLAN OF GATUN SPILLWAY ON PANAMA CANAL.

The floor of the spillway channel below the dam is 10 ft. above sea level, and the overflowing water, having fallen 75 ft., reaches it with a velocity of about 60 ft. per sec. Some stilling devices other than the effect of the convergence due to the curved crest appeared to be necessary to destroy the energy created by the fall, and a system of baffles was laid out in the floor immediately below the toe of the ogee.

While the model was available, it was possible to make tests to determine whether the proposed arrangement of baffles might be expected to accomplish the desired result. The model was on a scale of $\frac{3}{8}$ in. to 1 ft., the dam being made of concrete placed between wooden baffles cut to the proper section. The crest piers were of wood, grooved for wooden gates, so that the discharge could be regulated. The baffles were made of wood and fastened to the floor with screws to permit trials in different positions. A water supply was temporarily available from the drainage of the hydraulic fill then being placed in Gatun Dam, and a by-pass permitted the regulation of the flow so as to obtain any desired depth on the crest of the model dam.

FIG. 30.—MODEL DAM WITH OGEE SECTION.

FIG. 29.—MODEL OF STRAIGHT CREST DAM.

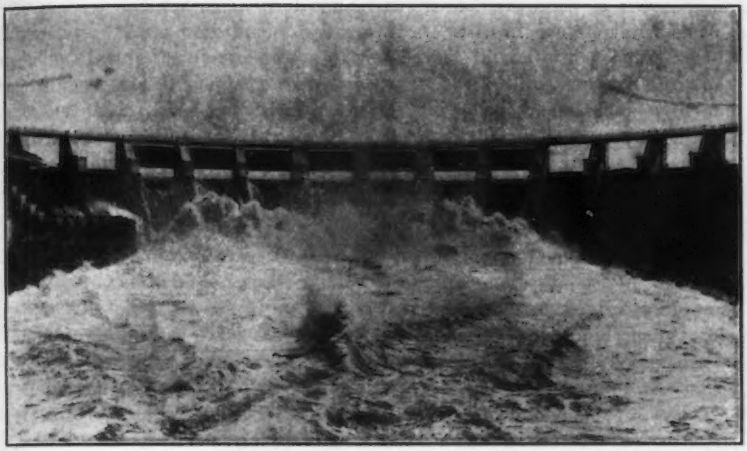


FIG. 31.—VIEW OF COMPLETED DAM.

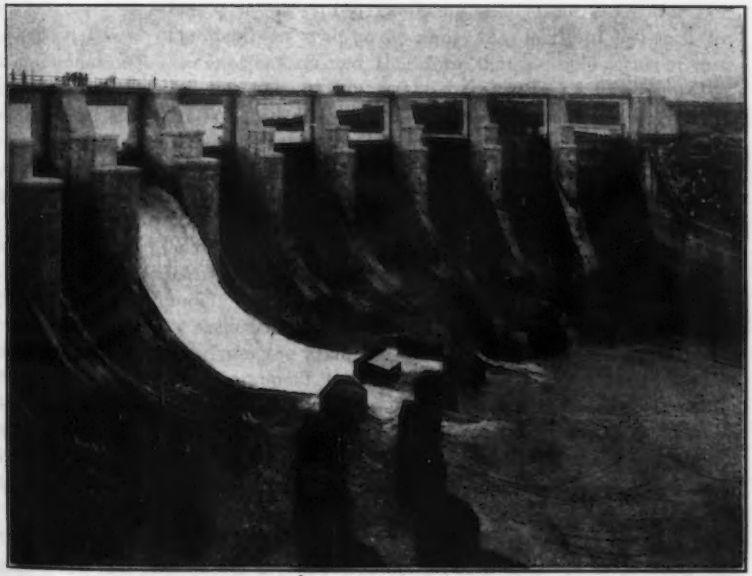
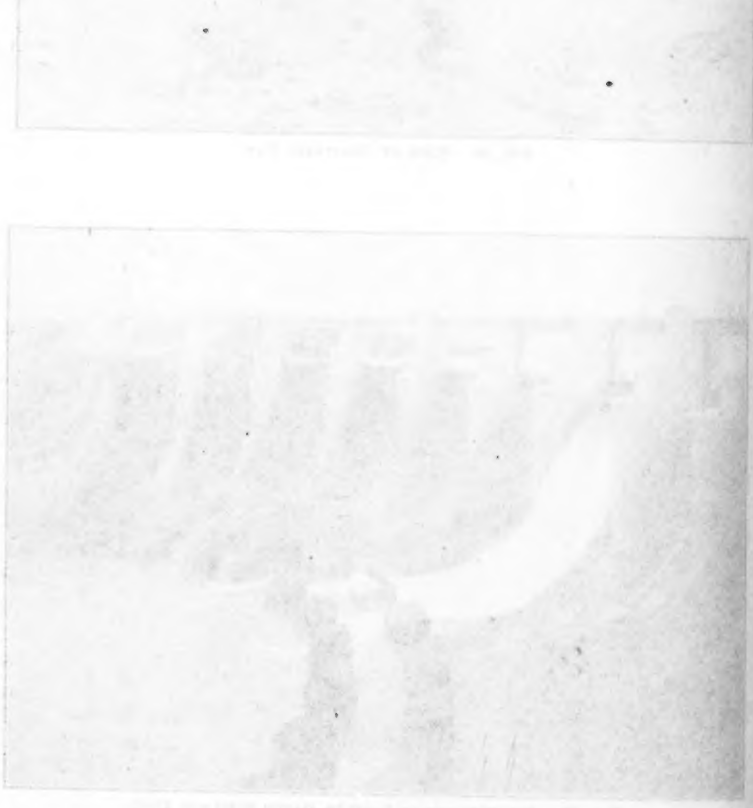


FIG. 32.—ARRANGEMENT OF BAFFLES, GATUN SPILLWAY DAM.



The time allowed for experiments was limited, because the water was very muddy, and the small pool behind the model was entirely silted up in a few days. Tests were made, however, first, with all contemplated baffles; second, with only the up-stream row of baffles; and, third, without baffles. Only approximate measurements were recorded, but the stilling effect of two rows of baffles, placed as shown in Fig. 28, was obviously greater than with any other arrangement that was tried. The effect of the baffles in establishing a standing wave and uniform flow in the channel is shown in Fig. 30, and a view of the completed dam in Fig. 31. The arrangement of the baffles in the actual structure is shown in Fig. 32.

B. E. TORPEN,* M. AM. SOC. C. E. (by letter).—The subject of scour below overflow dams or spillways is becoming more important as more and higher dams are built, and is of especial interest to hydraulic and designing engineers. The scour below the Wilson Dam,† and many other similar experiences with service dams, show the necessity of better design of spillway aprons. The type of spillway, and the treatment of the apron best suited to any special site are dependent on many factors, such as (1) safety, the factor of prime importance in all structures; (2) economy, a factor second only to safety in many cases; (3) type of dam, whether gravity, arch, earth, etc.; (4) architectural treatment; (5) hydraulic gradient of the stream; (6) geology; (7) topography; (8) climate; (9) proximity of power house or other structures; (10) percentage of time curve for certain flood flows; (11) depth of over-pour; and (12) depth to bed-rock for dam foundation. It cannot be expected, therefore, that a single series of tests on a special phase of this subject can establish the type of treatment best suited to all sites. It is hoped, however, that observed discharges during the next few years will demonstrate the similarity of actual conditions to the results of the tests, and prove beyond doubt the utility of models in illustrating new features for design.

In the Pit River tests the feature of causing the falling sheet of water to strike the mid-height of the splitter-piers and in a downward direction, seems especially effective. This was secured by placing a 15-ft. jump-off at the bucket of the dam.

The Pit dams are arched in plan, but of such long radius that no stability can be depended upon from the arch, the dam being of the full gravity section. The bottom of the jump-off, therefore, must be outside the full gravity section and all concrete in the bucket and jump-off is chargeable to the spillway, in addition to the massive piers. The total of this concrete is no small item and can not be overlooked when comparing types from the standpoint of economy.

In selecting the type of spillway for the Pit dams the authors have stated their reasons clearly. Their tests indicated the type best suited to meet those conditions, and now actual overflow will be the final test. It is believed that other unpublished data on tests of spillway models may add to the value of

* Hydr. Dept., Elec. Bond & Share Co., New York, N. Y.
† *Engineering News-Record*, February 3, 1927.

these discussions and with this in mind the writer presents, as briefly as possible, the results of a series of such tests for the Bull Run Dam constructed for the water supply of Portland, Ore.

The dam is 200 ft. high, of the concrete gravity type, and curved in plan as is common practice at sites slightly too wide for pure arches. The maximum flood may be 21 000 sec-ft., the free over-fall is 176 ft., and the minimum flow of the stream is about 160 sec-ft. The water-shed is only 75 sq. miles, but due to winter rains from 500 to 5 000 sec-ft. will spill over the dam half the time.

The topography is favorable for a spillway around the left end of the dam with a concrete apron to the edge of the bluff, 150 ft. above the river. The country rock is basaltic, in horizontal layers varying from 5 to 50 ft., and some flows are considerably altered. Some fear was entertained that the constant over-pour would erode this basalt and eventually require the spillway channel to be concreted to the water's edge, which made this location of the spillway rather costly. The alternate plan chosen was to carry the spillway over the central part of the dam to an apron, which extends down stream 150 ft. to protect the toe.

Toe Protection.—The question of further protecting the toe was next considered. Special treatment must be given the down-stream end of the apron also unless the velocity is reduced at this point. This has recently been illustrated at the Wilson Dam. To build heavy baffle-piers at the toe of the dam and concentrate the destruction of energy up to 350 000 h. p. directly over the toe (the most dangerous part of the dam foundation) seemed undesirable. This type of treatment also requires a "bucket" 15 ft. deep, and much extra concrete is necessary.

Baffle-piers at the bucket cause considerable spray at the toe of the dam and this was objectionable because the outlet-valve house is located immediately adjoining the spillway on one side and a future power-house setting is on the other. The spill water may at times carry large logs (in spite of boom protection) with enormous striking effect when hitting the nose of a baffle-pier. The conditions for destroying energy at the toe of the dam by a standing wave were unfavorable since the river gradient is so steep that water at all stages flows at a depth below the critical. A subsidiary dam below the main dam would have been required to create a deep pool. This, in turn, would have had to be protected from down-stream erosion.

The best plan seemed to be to carry the spillway sheet at a high velocity across the apron to a point 150 ft. down stream from the dam and there deflect the sheet upward so as to secure the maximum distance and spread of the striking area of the water mass and the river bed-rock. This plan was tested experimentally.

Model.—A 1 to 20 scale model of the spillway sections of the dam was constructed in the empty Reservoir No. 2 in the City of Portland, where adequate facilities for flow and control of water, head, etc., were possible. (See Fig. 33.) Flow over the model spillway was regulated by valves and gates and was accurately read on gauges to simulate certain discharges over the dam. Tests of various types and combinations of upward deflectors (see Fig. 34), were then

conducted for flows representing approximately 1 000, 3 500, 5 000, 10 000, 15 000, and 21 000 sec-ft.; measurements were recorded and characteristics of flow noted. (See Table 10.) An illustration of typical cases is shown in Fig. 35.

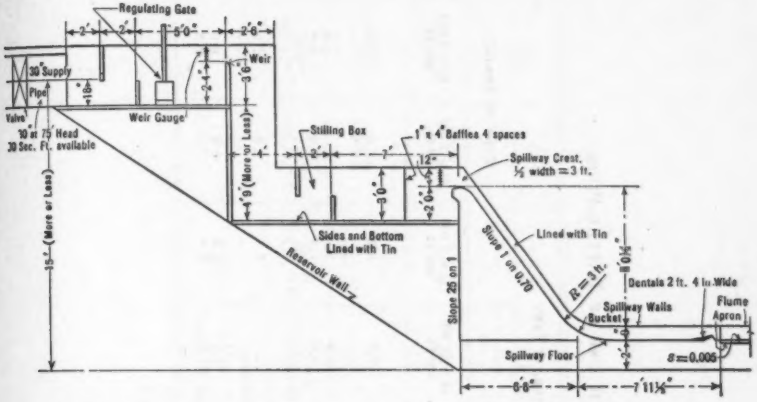


FIG. 33.—ONE-TWENTIETH SIZE MODEL OF SPILLWAY SECTION, BULL RUN DAM.

At first, curved throw-back baffle-piers, spaced alternately with deflectors (Fig. 34(*a*) and (*b*)), were tried in an attempt to destroy the energy, but the result was disappointing for the large flows because the water was thrown violently upward over the dentals to a height equivalent to 90 ft. (See Fig. 35.)

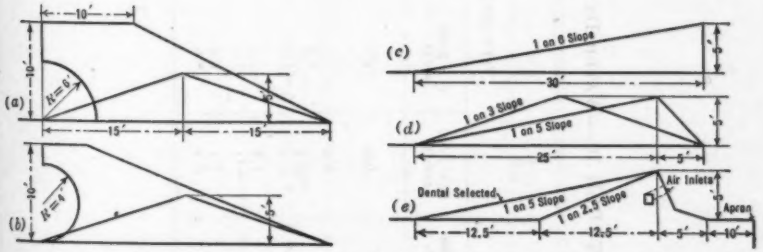


FIG. 34.—TYPICAL DENTALS TESTED WITH MODEL SPILLWAY, BULL RUN DAM.

Next a simple uptilt in the apron was tried. The end of the apron was raised 5 ft. in 30 ft. (Fig. 34 (*c*)), in the direction of flow. This served the purpose of lifting the fast traveling sheet into the air and preventing the rock from scouring close to the end of the apron. The disadvantage was that the attack of the falling sheet was concentrated and was still powerful although at a safe distance from the dam. To split this attack the apron was divided into dentals of different slopes (Fig. 34(*d*) and (*e*)), which spread the attack over a wide range by creating two separate parabolas. (See Fig. 36.) By varying

TABLE 10.—WATER DEPTHS OBSERVED WITH USE OF BAFFLE-PIERS.

Station.*	Depth of Water, in Feet.									
	Type (*a*) Dental.†						Type (*b*) Dental.†			
	$Q = 1\,000$ sec-ft.	$Q = 3\,500$ sec-ft.	$Q = 5\,000$ sec-ft.	$Q = 10\,000$ sec-ft.	$Q = 15\,000$ sec-ft.	$Q = 21\,000$ sec-ft.	$Q = 1\,000$ sec-ft.	$Q = 3\,500$ sec-ft.	$Q = 5\,000$ sec-ft.	$Q = 10\,000$ sec-ft.
Left:										
1 + 60	0.3	0.8	1.0	1.6	....	3.8	0.3	0.6	1.0	1.6
1 + 50	....	....	....	....	3.0	....	....	0.6	....	....
1 + 10	8.0	....	....	....	2.8	....	8.0	....	....	....
1 + 0	....	9.2	....	1.4	....	3.8	7.8	8.0	1.0	1.4
0 + 50	....	....	1.0	1.4	....	4.0	7.6	9.2	0.8	1.2
0 + 30	8.0	10.4	....	....	....	....	....	....	....	....
0 + 0	6 0	....	....	....	....	....	....	....	....	....
Right:										
0 + 50	6.0	7.6	....	....	....	....	5.8	7.8	7.2	...
1 + 0	....	....	....	....	....	....	....	....	....	4.0
1 + 50	....	....	....	....	....	....	....	....	....	....
2 + 0	5.8	7.4	8.6	10.0	9.0	11.0	6.2	8.2	8.6	....
2 + 25	5.2	6.2	8.0	9.4	10.0	11.0	7.8	7.0	7.8	9.2

* See Fig. 35.
† See Fig. 34.

TABLE 10.—(*Continued.*)

Station.*	Depth of Water, in Feet.									
	Type (*b*) Dental.†		Type (*c*) Dental.†						Type (*d*) Dental.†	
	Q = 15 000 sec-ft.	Q = 21 000 sec-ft.	Q = 1 000 sec-ft.	Q = 3 500 sec-ft.	Q = 5 000 sec-ft.	Q = 10 000 sec-ft.	Q = 15 000 sec-ft.	Q = 21 000 sec-ft.	Q = 1 000 sec-ft.	Q = 3 500 sec-ft.
Left:										
1 + 60	....	3.6	0.3	0.8	....	....	....	3.6	0.3	0.8
1 + 50	....	....	0.3	0.6	1.2	....	....	....	0.25	0.7
1 + 10	....	....	....	....	....	....	....	....	....	....
1 + 0	2.0	3.0	5.8	0.6	1.0	2.0	2.6	3.6	4.8	0.7
0 + 50	....	....	....	....	....	2.0	....	....	....	0.7
0 + 30	....	....	6.4	0.8	....	....	....	3.6	5.6	....
0 + 0	....	....	....	....	....	....	....	....	....	....
Right:										
0 + 50	....	....	6.4	6.4	....	....	....	....	6.0	5.8
1 + 0	....	....	....	....	....	....	....	....	....	....
1 + 50	....	....	....	....	....	....	....	....	....	....
2 + 0	5.8	9.0	5.5	7.2	8.4	6.0	8.0	6.0	6.0	7.4
2 + 25	10.2	11.4	5.0	6.6	8.0	9.0	9.0	10.0	5.6	7.0

* See Fig. 35.
† See Fig. 34.

TABLE 10.—(*Continued*)

Station.*	Depth of Water, in Feet.									
	Type (d) Dental.†				Type (e) Dental.†					
	Q = 5 000 sec-ft.	Q = 10 000 sec-ft.	Q = 15 000 sec-ft.	Q = 21 000 sec-ft.	Q = 1 000 sec-ft.	Q = 3 500 sec-ft.	Q = 5 000 sec-ft.	Q = 10 000 sec-ft.	Q = 15 000 sec-ft.	Q = 21 000 sec-ft.
Left:										
1 + 60	....	....	2.6	3.2	....	0.6	....	....	....	3.0
1 + 50	1.2	....	....	....	0.8	0.6	....	....	....	....
1 + 10	....	....	....	....	....	....	....	....	....	....
1 + 0	1.0	1.6	2.4	3.2	5.8	0.6	1.0	....	2.0	3.0
0 + 50	1.2	1.4	2.4	....	....	....	....	1.2	2.2	....
0 + 30	....	....	....	3.2	6.6	1.0	....	....	....	2.8
0 + 0	....	....	....	....	....	....	....	....	....	....
Right:										
0 + 50	....	....	....	....	5.8	5.0	....	....	....	....
1 + 0	....	....	....	....	....	....	....	....	....	....
1 + 50	....	....	....	....	....	....	....	2.0	....	....
2 + 0	8.8	4.4	4.0	5.0	6.0	7.2	1.8	....	3.2	5 0
2 + 25	7.8	9 6	8.8	9.0	5.8	7.2	8.6	7.6	5.0	....

*See Fig. 35.
†See Fig. 34.

these slopes the point of a falling jet may be regulated at will, within the limiting velocities of the overfalling sheet, which passes over the spillway floor and over the dentals without any appreciable loss of energy, and dissipates

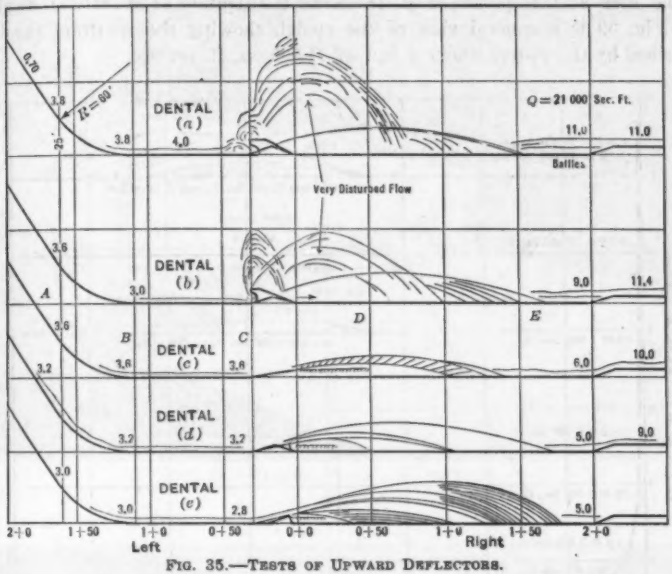


FIG. 35.—TESTS OF UPWARD DEFLECTORS.

itself in a well-scattered attack on the river channel far beyond the apron. Types (*d*) and (*e*) (Fig. 34), were both considered good. Type (*e*) was selected because of the ease of providing air inlets, although the necessity of these inlets was not apparent from the experiments.

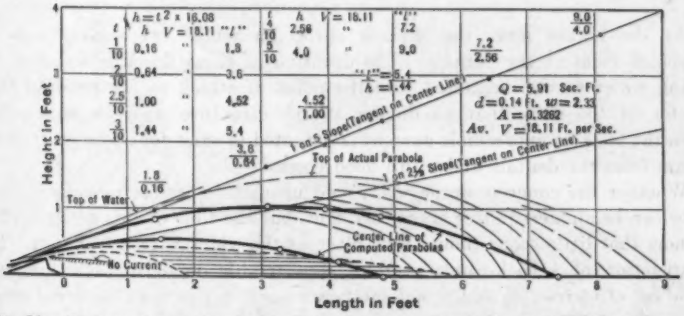


FIG. 36.—COMPARISON OF COMPUTED PARABOLAS AND FIELD MEASUREMENT OF ACTUAL PARABOLAS, SPILLWAY DENTAL TESTS FOR BULL RUN DAM.

From the "Percentage of Time Curve" (Fig. 37), it is evident that for 98% of the time the discharge over the spillway will be less than 3 500 sec-ft. which is required to sweep the apron at high velocity. During this long period the

discharge down the ogee slope will enter the pool on the spillway floor and form a small standing wave there. Over the dentals the over-pour will be steady and a 10-ft. apron is provided in Type (*e*) (Fig. 34) to prevent erosion of the rock at this point. Fig. 38 shows the dentals (Fig. 34(*d*)) in place and Fig. 39 is a general view of the model, showing the resultant parabola described by the waters under a flow of 21 000 cu. ft. per sec.

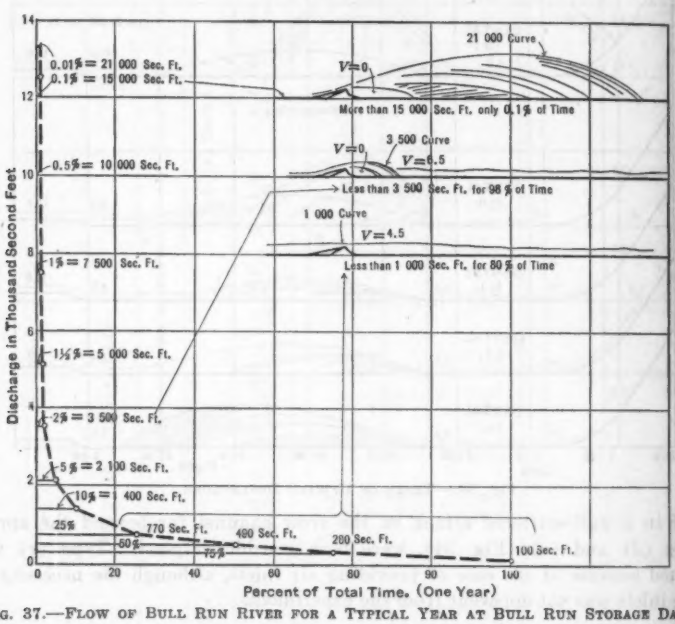


FIG. 37.—FLOW OF BULL RUN RIVER FOR A TYPICAL YEAR AT BULL RUN STORAGE DAM.

At the higher flows the shallow apron bucket is swept clear and the parabolas form at the dentals. The duration of these floods is so short, the falling water so well scattered, and the point of attack so far removed from the toe of the dam, that no danger to the structure need be anticipated. From the tests no appreciable current is expected to exist for about 40 ft. down stream from the dentals under full flood flows.

Whether the concrete apron will stand up under the fast moving sheet of water can be answered after actual overflow, but the time of this action will be so short that little scour should take place as the water is free from silt. The construction of a back-water pond of considerable depth for a stilling-pool would be, of course, an additional safety measure, but is not considered necessary. The falling parabolas may excavate a pool, in time, which will serve the same purpose. The dental piers, as constructed, are practically identical with Model (*e*) (Fig. 34), except that the down-stream portion is excavated 10 ft. into the solid rock and poured as one piece with the piers and the whole anchored into the rock with steel rods. The advantages of this type of spill-

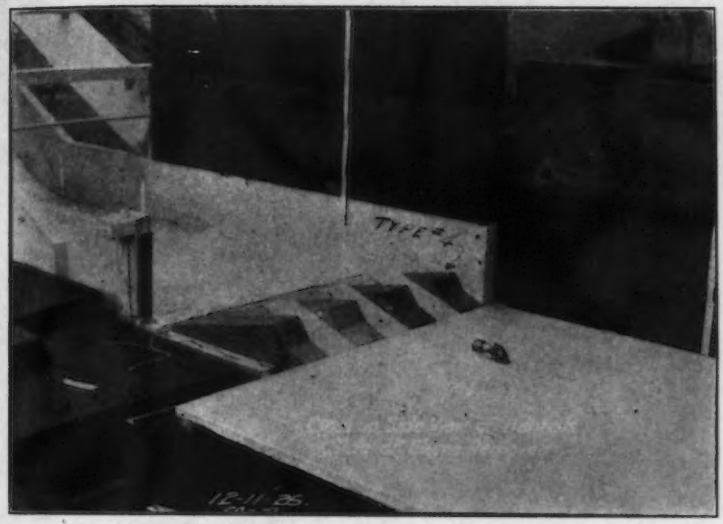


FIG. 38.—BAFFLES, TYPE (*d*), IN PLACE.

FIG. 39.—SHAPE OF PARABOLA, AS SEEN BY REMOVING THE SIDE OF THE FLUME.

way are simplicity, economy, and safety. For greater economy the fore-apron could be shortened considerably. The cost of the dentals is very low as compared to deep buckets, jump-offs, depressed aprons, or stilling-pools.

The curve at the bucket was first tested for a short radius of 10 ft., but the rebound of the overfalling sheet was very pronounced at the larger flows. A radius of 100 ft. was next tried and gave very smooth delivery, but at quite an expense in extra concrete over a short radius. A radius of 50 ft. was next tried, and this gave fairly good results. For additional safety, a radius of 60 ft. was chosen.

These tests were carried out under the personal direction of D. C. Henny, M. Am. Soc. C. E., Consulting Engineer, and Ben S. Morrow, Assoc. M. Am. Soc. C. E., Chief Engineer, of the Bull Run Dam.

OREN REED,* ASSOC. M. AM. SOC. C. E. (by letter).—Although a great number of problems, such as those presented by the author, are difficult, or impossible, to solve by mathematics, opportunity is seldom given to attempt their solution by experimental observation. In Europe, on the other hand, extensive laboratory and field experiments are run on models of many important projects, before the final designs are made.† This is particularly true of hydraulic problems.

Difficult foundation conditions introduce a special problem which, unless properly taken care of, forms a great hazard for a dam. The experiments described by the authors have made it possible to insure that the over-pour will leave the concrete apron at a low velocity under the most unfavorable conditions.

The design adopted is very similar to the energy-controlling device at the Gatun Dam. However, the baffle-piers at the Gatun Dam are probably secondary in importance for destroying the energy of the spill. The spillway is in the form of a circular arc, 808 ft. in length along the crest. The discharge converges to a concrete channel, 285 ft. wide. The energy of the converging stream will partly neutralize itself, while the baffle-piers will aid materially. The Gatun Spillway has proved its effectiveness to pass high flood flows.

Before the flood control dams for the Miami Conservancy District were built in the Miami Valley, Ohio, many experiments were made to determine the best form for several features, including the outlet and spillway structures.‡ The outflow from certain of these dams may become large and the banks of the channels below the outlets are of soil, which is easily eroded. After thorough trial the use of stilling-pools was adopted as the most practicable method of controlling the energy of the outflow. The pool was constructed so that a hydraulic jump occurred through a wide range of discharge and tail-water conditions. The position of the jump was controlled within narrow limits by sloping the floor downward from the outlets. For the conditions below the Miami Valley dams, the use of baffle-piers was found to give

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† "European River-and-Harbor Laboratories Revisited," by John R. Freeman, Past-President, Am. Soc. C. E., *Engineering News-Record*, Vol. 99, December 1, 1927.

‡ Technical Report, Pt. III, Miami Conservancy District.

poor results. The flow on the models was not stable under moderate changes of discharge and tail-water level, or in changes of the form of the structures. It was considered that the analogy between the models and the full-sized structures was inconclusive.

Model experiments may often be carried out on complete plants. This was done in 1919 on a 1 to 25 scale model of the Solbergfos Power Project in Norway. At Solbergfos, the Glommen River is controlled by a gravity dam with three roller-gates for flood regulation. The power house is parallel to the stream, and the up-stream end forms part of the dam. The reaction units, thirteen in number, take water from an open forebay and discharge into the river just below the dam. By these experiments an arrangement was determined which gave the least interference of the spillway over-flow with the discharge from the power house.

The conclusions from model experiments should be mainly qualitative because, even under the most favorable conditions, the water is in a very turbulent state, and any quantitative measurements will be inconclusive. Caution must be observed because slight peculiarities in the model might correspond to great irregularities in the complete structure. The observation of tendencies is, however, of great importance. More problems should be attacked by this method, and it is hoped that the future will see a marked change in attitude toward such research.

HAROLD K. FOX,* M. AM. SOC. C. E. (by letter)—A real experiment such as the authors have conducted and have so ably presented, always adds something worth while to engineering knowledge. The writer visited the project during construction and, therefore, can fully realize the problems encountered.

A description of an emergency job which, at the same time was an experiment, is prompted by this paper. The problem is not in connection with a dam, but is one of the same nature, namely, destroying the energy of water to prevent erosion.

A concrete lined ditch, 3½ miles long, with a capacity of 100 sec-ft., was constructed to divert water into a reservoir. The discharge end terminated above the lake level and about ½ mile from the shore. Water was discharged into a ravine leading to the lake, and the problem was to prevent undercutting at the end of the ditch, as the material was a deep deposit of red clay and decomposed granite which eroded very easily. After some study, it was finally decided to connect the end of the ditch to a steel pipe in order that the water might be conveyed down the steepest part of the ravine and then be made to flow in the channel. At the end of the pipe where the velocity was 35 ft. per sec., a very crude wooden structure was built for the purpose of destroying energy and transferring the water to the ground surface at a low velocity. Fig. 40 shows the method of splitting the water as it left the pipe and creating a "boil" within a chamber. It was found necessary to build a roof over this chamber and weight it down with rocks as the spray got behind the retaining walls on the sides and started to cut. From this chamber, the water flowed over a pile of rock which further reduced the velocity and then, finally, over

* Supt. of Constr., San Joaquin Light & Power Corporation, Fresno, Calif.

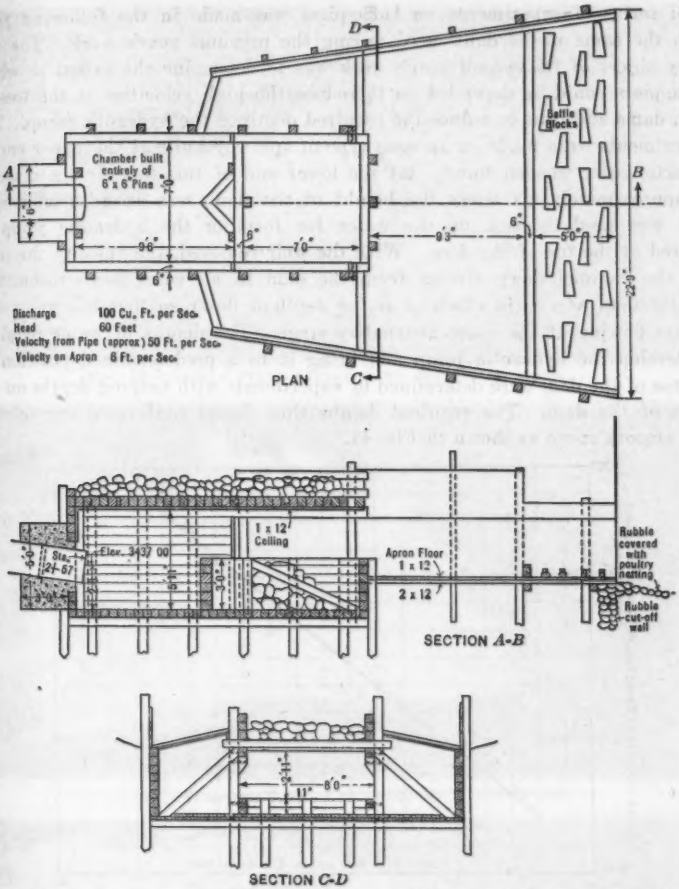


FIG. 40.—PLAN AND SECTIONS, ENERGY DISSIPATING CHAMBER, BROWN'S CREEK DITCH.

a series of baffles, and left the platform with a velocity of about 6 ft. per sec. The structure has stood for several years without repairs and apparently will give service for several more years.

ROBERT F. EWALD,* ASSOC. M. AM. SOC. C. E. (by letter).—In 1915 the writer obtained some data relative to baffle-piers at the toe of high over-fall dams. This information is presented herewith in the hope that it will prove to be a worth-while addition to the general store of knowledge on the subject to which the authors have contributed so much.

The data were obtained as part of a rather elaborate series of experiments, begun in 1914, on the hydraulic jump in relation to high over-fall dams. A

* Asst. Engr., The Aluminum Co. of America, Pittsburgh, Pa.

brief series of experiments on baffle-piers was made in the following year, with the same model dams used during the previous year's work. The primary object of the second year's work was to determine the extent to which baffle-piers could be depended on to reduce the high velocities at the toes of high dams and thus to reduce the required depth of the hydraulic jump. The experiments were made on an ogee type of spillway built at the upper end of a rectangular wooden flume. At the lower end of this flume, at a distance of approximately six times the height of the dam, was an adjustable weir that was used to back up the water for forming the hydraulic jump as desired at the toe of the dam. With the weir removed, the velocity throughout the channel down stream from the dam in all cases was considerably greater than $\sqrt{g\,d}$ (in which, d is the depth of flow), so that the water was always flowing at the lower alternative stage. The depths of water required to develop the hydraulic jump and bring it to a predetermined position at the toe of the dam were determined by experiments with varying depths on the crest of the dam. The required depths thus found conformed very closely to a smooth curve as shown in Fig. 41.

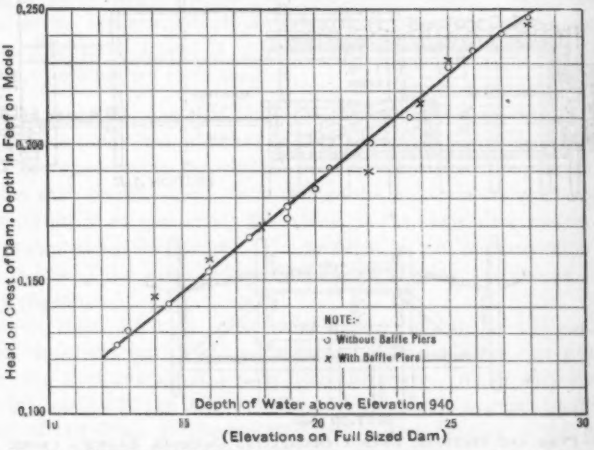


FIG. 41.—DEPTHS OF WATER REQUIRED TO BRING JUMP TO TOE OF DAM WITH AND WITHOUT BAFFLE-PIERS.

The problem arose as to reducing the velocities sufficiently to form the jump and one of the suggestions was the use of baffle-piers. It seemed that if they possessed any pronounced efficiency in absorbing the high over-fall velocities, such efficiency could be determined by noting the reduced depth of water required to develop the hydraulic jump. In following out this idea, a line of baffle-piers was attached to the bucket of the dam, as shown in Fig. 42. These baffles were of the same height as the estimated maximum depth of water at the point where the velocity was greatest, and were staggered so that the deflected jets presumably would "interfere" with each other to the greatest possible degree.

FIG. 42.—BAFFLE-PIERS ATTACHED TO BUCKET OF MODEL.

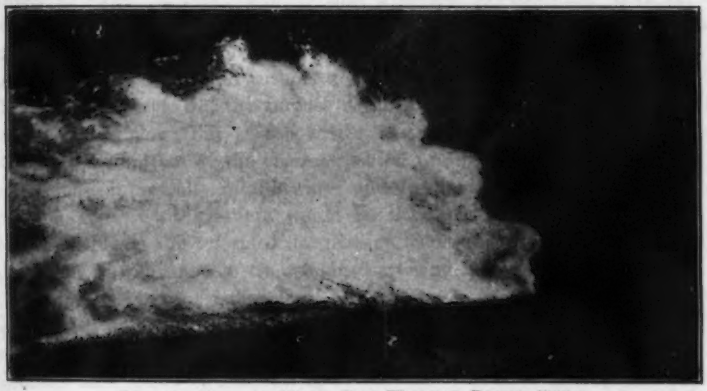


FIG. 43.—VIEW OF MODEL DAM WITHOUT BAFFLE-PIERS.

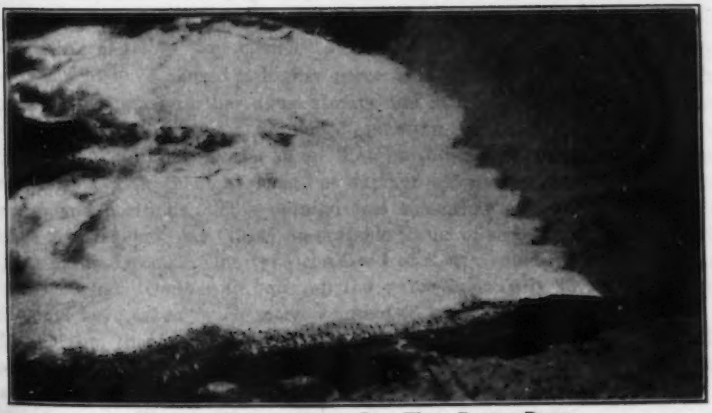


FIG. 44.—VIEW OF MODEL DAM WITH BAFFLE-PIERS.

Another series of experiments was run, and, as before, the head on the crest of the dam was obtained as well as the depth of water (in the down-stream channel), that was required to bring the beginning of the jump to the predetermined position at the toe of the dam. It was a rather difficult matter to establish the beginning of the jump with any degree of accuracy and a somewhat indirect method was required. The procedure was to establish the jump clearly at the extreme down-stream end, then to build up the pool depth, watching the movement of the jump up stream, until its beginning was lost in the disturbed flow around the piers. By relating the increment of movement to the increment in depth, it was possible to make a fair estimate of the depth required to bring the beginning of the required jump to position. These estimated depths were plotted on the curve, previously developed (see Fig. 41) and since they fell indiscriminately above and below, it appeared to the experimenters that the energy dispelled by the piers was relatively so small in proportion to that absorbed by the jump that further experiments along this line would not prove of particular profit. Comparative photographs were taken, similar to those shown in Figs. 43 and 44, to record conditions with and without the baffles. There was little question but that, with the baffles in, the surface action of the water was much smoother than with them out, but as the writer was primarily interested in the conditions at the bottom of the channel, which presumably were reflected by the net decrease in velocity of the water and freedom from its up-stream movement, it appeared that there would be little practical gain in the use of baffles. In fact, they seemed to possess a practical disadvantage in that they would be subjected to tremendous battering action and must be repaired from time to time. Furthermore, unless they were very skillfully designed and placed, and the gates on the dam correctly operated, water would be sprayed everywhere. This was a positive detriment with the power-house locations then under consideration.

It is apparent from Figs. 42, 43 and 44, that the internal interference caused by these baffles is less than that obtained in the Pit River experiments; yet having had the "first chance" at the high velocities, these baffles apparently should have caused a higher percentage of reduction in velocity than piers located much farther down stream and nearer the middle of the jump. Unfortunately, the writer developed such an unfavorable opinion of the baffles as an adjunct to the jump that he did not attempt any systematic experiments with them at locations farther down stream or with different shapes and positions. This opinion was somewhat affected by experiments made the previous year (1914) on suitable forms of secondary dams for developing cushion pools. One type consisted of heavy square-nosed piers spaced closely enough together to back up the water sufficiently to form the jump. It would have taken much more concrete to build piers of sufficient size for this purpose than for a dam of the straight ogee type. Having tried piers at both ends of the jump, the middle did not appear very promising and was completely neglected.

The principal requisites for good action in the hydraulic jump were clearly established during the experiments. These principles, with some additions, have been clearly expressed* once more by Sherman M. Woodward, M. Am. Soc. C. E. One of the requisites stated by Professor Woodward is:

"That if the slope of the jump off be made greater than one in three, experiments indicate that the certainty and effectiveness of the jump in destroying energy are perceptibly reduced. If the slope is made as steep as 45°, the usefulness of the jump is mostly lost".

It appears to the writer that the steep slope of the down-stream end of the bucket used on the Pit River dams must interfere very seriously with the development of the jump in its desired form. In the Pit River experiments, the inability to secure satisfactory action of the jump without the piers may have been due in large measure to the steep slope of the bucket and the failure to place the control weir far enough down stream. Dam No. 3 was built with an apron extending 150 ft. down stream from the bucket. It would have been interesting to know the results that would have been obtained by lowering the bucket until its lower edge was nearly tangent to the apron, and then placing a single line of piers about 100 ft. from the edge of the bucket instead of 47 ft. to the more distant pier. The baffles then would act as a control weir. The end results probably would have been nearly the same as those secured by the authors, but the baffle-piers would then have been saved from the tremendous battering to which they are now subject.

Any one who has watched and experimented with the hydraulic jump as developed under suitable conditions cannot avoid marveling at its efficiency in absorbing high velocities without effect on the channel, and the conviction grows that in its action lies the secret of the success of most over-fall dams, otherwise many structures standing to-day would have disappeared as the result of erosive action. Whether or not artificial construction is really necessary to develop the jump in its most efficient form, depends entirely on local conditions. Usually, there is no serious objection to permitting scour, providing the down-stream channel conditions and the design of the bucket are such as to prevent eddies scouring backward toward the toe of the dam. Eventually, Nature will scour a pool sufficiently deep to permit the development of the jump in its most efficient form and the scouring action will then cease.

Where scour in the immediate vicinity of the toe is undesirable, it appears entirely logical to construct a smooth apron in front of the dam just as was done at Pit River, carrying it far enough down stream so that the hole that will be scoured at the end of the apron will not be too close to the dam; then "letting it go at that". It is true that some of the apron may be lost in the hole that will be formed at its end, but it is equally certain that eventually a natural cushion pool will develop and further scouring will cease. It is only a question of determining the size of pool required and making the apron long enough so that, assuming the rock to have been cut backward

* In a letter to *Engineering News-Record*, December 15, 1927, entitled "Toe Erosion Below Overfall Dams."

under the apron, the final pool will be of the required dimensions and far enough away from the dam to avoid trouble. Of course, where eroded material may land in tail-races, such procedure is not permissible, and provision must be made for an artificial pool. Very high over-fall dams are generally founded on exceptionally good rock and long aprons usually are not necessary. The very hardest rock will be eroded, and at a surprisingly fast rate in many cases, but such erosive action through proper design of the bucket can be forced to take place at a considerable distance from the dam and dangerous "back lash" eliminated.

Just what constitutes "proper design" has not yet been clearly established, but that it is attainable is illustrated by two high dams located on Little Tennessee and Yadkin Rivers in North Carolina, designated as the Cheoah and Narrows Dams. At Cheoah Dam, in which the drop from the spillway crest to normal water level is 175 ft., the lip of the bucket is about 5 ft. below the normal water level, and there is a natural pool about 30 ft. deep at low water, the depth rapidly increasing with higher stages. This dam has had many floods over its crest with 19 ft. of water passing through some gates. Ever since the last and most severe flood in 1920, which caused considerable scour at some distance from the dam, gravel and loose stone have actually been piled against the toe of the dam below the lip of the bucket, indicating complete absence of erosive eddies at the toe. At the Narrows Dam, the drop from the crest of the dam to normal water level is 170 ft.; the lip of the bucket is at normal tail-water level, and the stream in front of the dam is relatively shallow. Considerable scour has taken place much closer to the dam and in a few spots the rock has been taken out directly in front of the bucket for a depth of 6 ft. In no place has loose material remained against the toe of the dam below the lip of the bucket. The rock at the Narrows Dam is considerably more friable than that at the Cheoah Dam, but this does not explain why loose material will pile against the bucket at one dam and not at the other.

The differences in conditions must be explained by (1) differences in natural pool depth; (2) differences in position of lip of bucket (designated "jump off" by Professor Woodward) relative to bottom of pool; and (3) whether or not the water is permitted to flow directly away from the dam or is forced by obstructions into major eddies in the cushion pool. (By obstructions is meant large masses of rock or heavy structures projecting into the thread of the current.) Major eddies may also be caused by jets of water directed into the pool at large angles to the main flow down the dam. At the Cheoah Dam the natural pool is three times as deep as at the Narrows Dam, and the relative position of the lip of the bucket to the bottom of the pool is much lower than in the second. At Cheoah, it has been possible to turn water into the pool in a fairly uniform direction, and the pool is relatively free from major obstructions, whereas at Narrows some water has been turned into the pool at a fairly large angle to the main flow and rock obstructions also develop large eddies.

To the writer it appears that the ideal method of absorbing kinetic energy of water at over-fall dams is to take the water through the entire drop over perfectly smooth surfaces into cushion pools, functioning in accordance with the laws governing the formation of the jump, and avoiding all obstacles to the free and direct flow of the water away from the dam, with the exception of those primarily designed to increase the depth of water at the toe of the dam, providing they are far enough away from the dam to permit the formation of the jump. For economic reasons scouring usually can be permitted, care being taken to predetermine the probable ultimate dimension of the pool and to see that it is eroded at a reasonably safe distance from the dam. Ultimately, by the formation of the jump under ideal conditions, the erosive action will cease, and no further trouble may be expected so long as the bucket is maintained in its proper condition.

F. W. SCHEIDENHELM,* M. AM. SOC. C. E. (by letter).—The authors' statements and their description of the tests indicate that observation of the surface of the water and photographic study were largely relied upon as a basis for comparing the various experiments. The importance attached by the authors to surface appearance is further emphasized by their characterization of the results of Experiment No. 13 which, in model form, approximated the final construction. However, the writer's experience in experimentation by means of models, as to energy destruction and scour at the foot of spillways of dams, indicates strongly that the appearance of the surface of the water flowing from the foot of the spillway is not a reliable criterion regarding the effect on the stream bed. After all, the most reliable criterion for comparison of the various means of preventing scour is the action on the stream bed itself.

Experiments made under the writer's direction at the hydraulic laboratory of Cornell University indicated that the relative height of tail-water is of great, even of controlling, importance. In the model dam tests for determining the design of Pit River Dam No. 3, there was provided a "back-water control", as shown in Figs. 2, 3, and 4. Apparently, this control or block (perhaps of wood in the model), was not embodied in the final or full-sized structure (see Fig. 1). However, the writer's experiments indicate that the omission, in the full-sized structure, of a control located so close to the foot of the spillway would result in a performance differing from that in the model. In particular, he suspects that there would result somewhat greater scour of the stream bed than if the equivalent of such a block or control were embodied in the final construction.

In the case of the model experiments for the dam of lesser height (No. 4 Dam) a so-called "two-level" apron was tested and a similar apron was embodied in the final construction. The writer is inclined to the belief that the "back-water control" (again, apparently, a block in the model), would be of greater significance in the case of the higher apron, which is shown at the far side of the model in Figs. 17, 26, and 27. The near, or lower, side of the apron would seem, in effect, to involve a stilling-pool, terminating in the

* Cons. Engr. (Mead & Scheidenhelm), New York, N. Y.

equivalent of a solid transverse baffle-wall nearly 14 ft. high and located about 70 ft. down stream from the vertical face of the spillway bucket (both dimensions are in terms of the full-sized structure). The writer has not overlooked the authors' statement that "the difference in the final result, as indicated by the velocity in the river channel below, was, as nearly as could be judged, uniform across the width." However, here, too, one may infer that judgment was based primarily on surface appearance, which may be misleading. Moreover, even the authors state that "the 'killing' of velocity was accomplished with less commotion on the deep side than on the shallow."

A study of Figs. 26 and 27 indicates that the right or far bucket was swept clear, whereas the left or near bucket, namely, that opposite the equivalent of a "stilling pool," was covered by back-roll. This is, to some extent, a measure of the effectiveness of a given arrangement for the prevention of scour.

Inasmuch as the experiments described by the authors apparently did not involve simulation of the stream bed, nor any observation of scour, it is the more desirable and important that the results on the stream bed should be observed at the foot of the actual structures.

RALPH L. PARSHALL,* ASSOC. M. AM. SOC. C. E. (by letter).—The control of scour below dams resulting from the impact of the water passing over the spillway, presents a problem of great importance in the design of such structures, for the reason that the erosion, unless prevented, comprises a potent element of failure.

In April, 1917, the writer designed and built a concrete spillway for a small reservoir near Windsor, Colo. (Fig. 45). This spillway, which is essentially a chute, has a width of 50 ft. at the crest, narrowing to 8 ft. in a distance of 75 ft., with a uniform grade or drop of 8.5 ft. in that distance. The bottom width is increased to 24 ft. in the next 10 ft. of length. The axis of the crest of the spillway forms an angle of 80° with the north side of the channel which is straight, while the south side curves in plan from a width of 50 ft. to 8 ft., as stated. The side slopes are 1 on 1. The depth of this converging section is 2 ft. The side walls at the lower end are 4 ft. high.

A vertical drop of 4 ft. occurs in the grade at the lower end of this chute. Across the channel at the drop is a vertical concrete baffle-wall, 4 ft. high and 8 in. thick, joined to the 1 on 1 side slope of this basin and set 1.5 ft. above the floor line. This vertical wall is supported by three equally spaced buttress walls on the lower face of this transverse baffle-wall; 5 ft. down stream is a transverse parallel wall, 1.5 ft. high, on the floor of the basin. A hole is left at the floor line of this low wall for drainage purposes. Most of these features are shown in Fig. 45.

These transverse walls are for dissipating the energy of the stream by impact. The lower or secondary wall creates a water cushion and chamber for the discharge as it passes over the end of the approach channel.

This spillway was built largely to forestall an emergency caused by a sudden rise of the water surface within the small reservoir, such as might be

* Irrig. Engr., U. S. Dept. of Agriculture, Fort Collins, Colo.

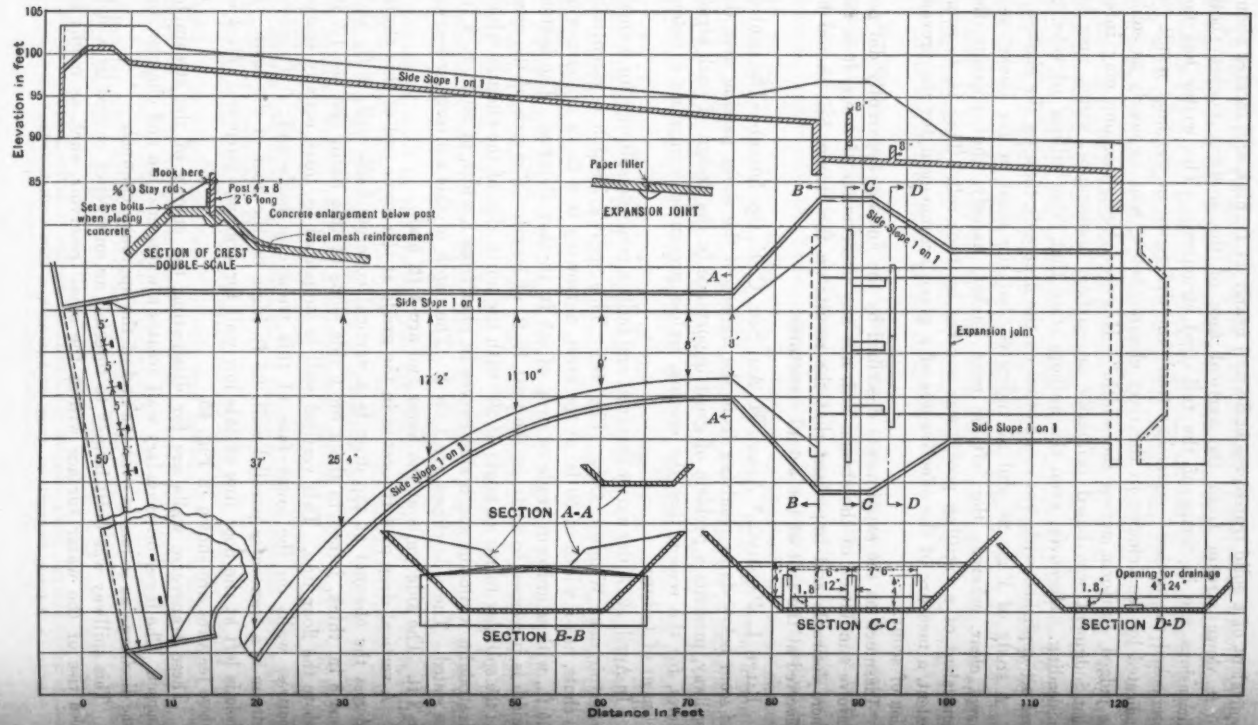


FIG. 45.—PLAN OF CONCRETE SPILLWAY FOR RESERVOIR NEAR WINDSOR, COLO.

FIG. 46.—HALLIGAN DAM, SHOWING SPILLWAY IN ACTION, WITH A DISCHARGE OF 700 SECOND-FEET, JUNE 24, 1917.

FIG. 47.—CLOSER VIEW OF CONDITIONS AT HALLIGAN DAM, SHOWN IN FIG. 46.

FIG. 45.—PLAN OF CONCRETE SPILLWAY FOR RESERVOIR NEAR WINDSOR, COLO.

Distance in Feet

caused by a freshet in the stream. The writer has never seen this structure in action, but has been informed that for relatively small discharges it appears to function properly.

An interesting example of scour below dams is shown at the Halligan Dam, on the North Fork of Cache la Poudre River, about 35 miles northwest of Fort Collins, Colo.* This structure (Fig. 46) is of reinforced cyclopean concrete. The width of the base is 51 ft.; the extreme height, 94 ft.; and the length, along the top of the dam, 350 ft. The dam, completed in May, 1910, is of the arch type with a constant radius of 324 ft. The foundation is a hard seamy granite mixed with red talc formation. Table 11 shows the flow over the spillway for the period, 1919 to 1926, inclusive.

TABLE 11.—DISCHARGE OVER HALLIGAN SPILLWAY.

Year.	SPILLWAY IN ACTION.		DISCHARGE, IN SECOND-FEET.	
	Period.	Days.	Maximum.	Average.
1919	None	None	0	0
1920	May-June-July	57	850	420
1921	May-June	21	640	240
1922	None	None	0	0
1923	May-June-July	44	2 740	490
1924	May-June	29	710	385
1925	None	None	0	0
1926	April-May-June	40	570	170

These data were taken from a record of reservoir gauges showing the depth of water in tenths of feet above the crest elevation. The discharge was computed by the formula, $Q = C\,L\,H^{\frac{3}{2}}$, in which C is assumed to be 3.5. The length of crest is 110 ft. Since the completion of the dam there has been reported, on more or less reliable information, a depth of 4.7 ft. on the crest of the spillway. For this head the discharge would have been approximately 3 900 sec-ft.

After a period of about 18 years of service, there appears to be no scour below this dam. As indicated, the extent of the discharge over the crest of spillway is dependent on the annual run-off from the water-shed above the reservoir. In some years there has been no flow over the dam, while for others the maximum has been nearly 4 000 sec-ft. These maxima, as shown, occur for very short periods. Fig. 46 shows a discharge of about 700 sec-ft. passing the crest, while Fig. 47 gives a more detailed view below the dam. Both views indicate, when compared with photographs taken at the completion of the dam,* that there has been very little if any increase of scour, because of the nature of the formation.

H. W. DENNIS,† M. AM. SOC. C. E. (by letter).—There is a well-defined growing tendency to take advantage of experiments on models as a guide in the design of major structures, and the authors have indicated such an appli-

* "The Halligan Dam; A Reinforced Masonry Structure," by G. N. Houston, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXV (1912), p. 112.

† Chf. Civ. Engr., Southern California Edison Co., Los Angeles, Calif.

cation and submitted their results for the benefit of the Engineering Profession. They have called attention to the question as to the quantitative interpretation of the results, although it is clear that qualitatively they are valuable.

The following may be a contribution to the subject from the standpoint of the quantitative interpretation of the hydraulic scouring action to be expected when a large quantity of water is discharged into an uncompacted material. In this case, it was the dump from the Florence Lake Tunnel, constructed by the Southern California Edison Company for conveying water in a comparatively large quantity from the Florence Lake Reservoir to the Huntington Lake Reservoir of that Company.

Although the outlet portal of the tunnel was excavated in earth and required a concrete lining for a distance of about 800 ft., the remainder of the tunnel was excavated in solid granite, and the material was deposited, as was convenient, near the portal of this tunnel. The elevation at the portal is 7 000 ft. above sea level, and Huntington Lake, when full, is at Elevation 6 950. Accordingly, provision was required that the water should be permitted to discharge without undercutting the tunnel portal. With an anticipated maximum discharge from the tunnel of about 2 300 sec-ft., a steel pipe 12 ft. in diameter was inserted in the end of the concrete lining and was constructed on a 7.85% grade for a distance of 258 ft., so as to bring the actual outlet at an elevation of 6 980 ft., where the pipe was supported by a concrete head-wall. The diversion into the tunnel is controlled by gates, and the first operation through this outlet structure afforded opportunity to take measurements of the scour which would be produced by flow up to a maximum available capacity. At the time of the experiment, this was limited to 1 875 sec-ft., due to a reduced elevation of the upper reservoir.

Contour surveys were made before any water was diverted and also after the maximum flow (1 875 sec-ft.) had been allowed to run for several hours, at the end of which time no further apparent scour of the channel was taking place at the outlet structure. The outlet structure, under conditions of minimum and maximum flow, is shown in Figs. 48 and 49, respectively. Fig. 50 shows the plan and profile, including contours so taken.

Fig. 50 shows a large rock located almost exactly on the center line of the pipe and about 55 ft. from the outlet. The presence of this rock was unknown at the time of laying out the work, and, of course, it functioned somewhat as a baffle and assisted in destroying the energy of the discharging water. The natural weir created below this pipe was such that, when no water was discharging, the water stood at Elevation 6 978, whereas, during the operation, the general elevation of the water on this weir was 6 987 ft.

No horizontal or vertical curve occurred in the pipe near the outlet, and accordingly the general slope of the jet corresponded to the condition recited by the authors as obtaining at Pit No. 3 Dam, in that the direction of flow is downward rather than truly horizontal.

The size of the material which remained as a natural lining of the pool at the conclusion of the observation would be roughly classed as "one-man stone".

FIG. 48.—VIEW OF OUTLET STRUCTURE.

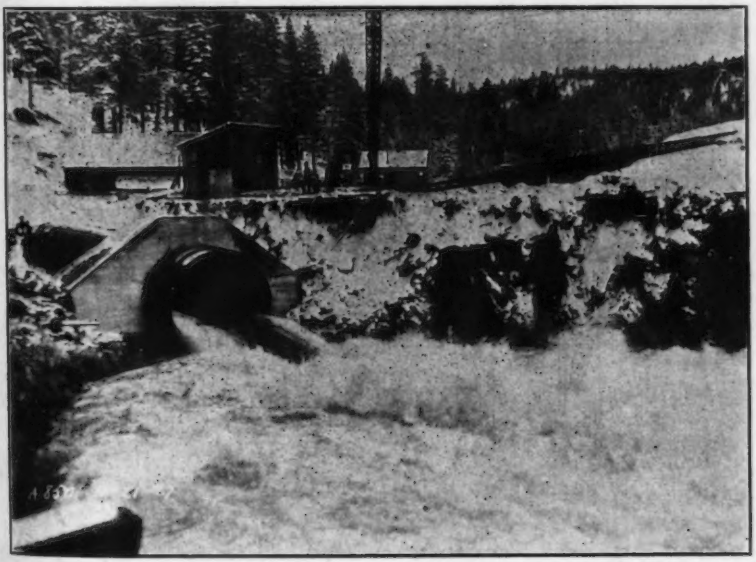
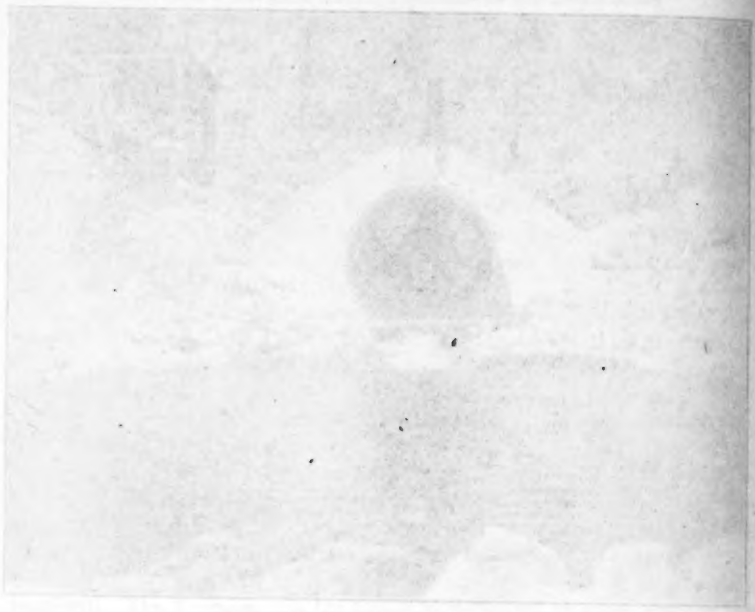


FIG. 49.—VIEW SHOWING MAXIMUM FLOW OF 1 875 CUBIC FEET PER SECOND.



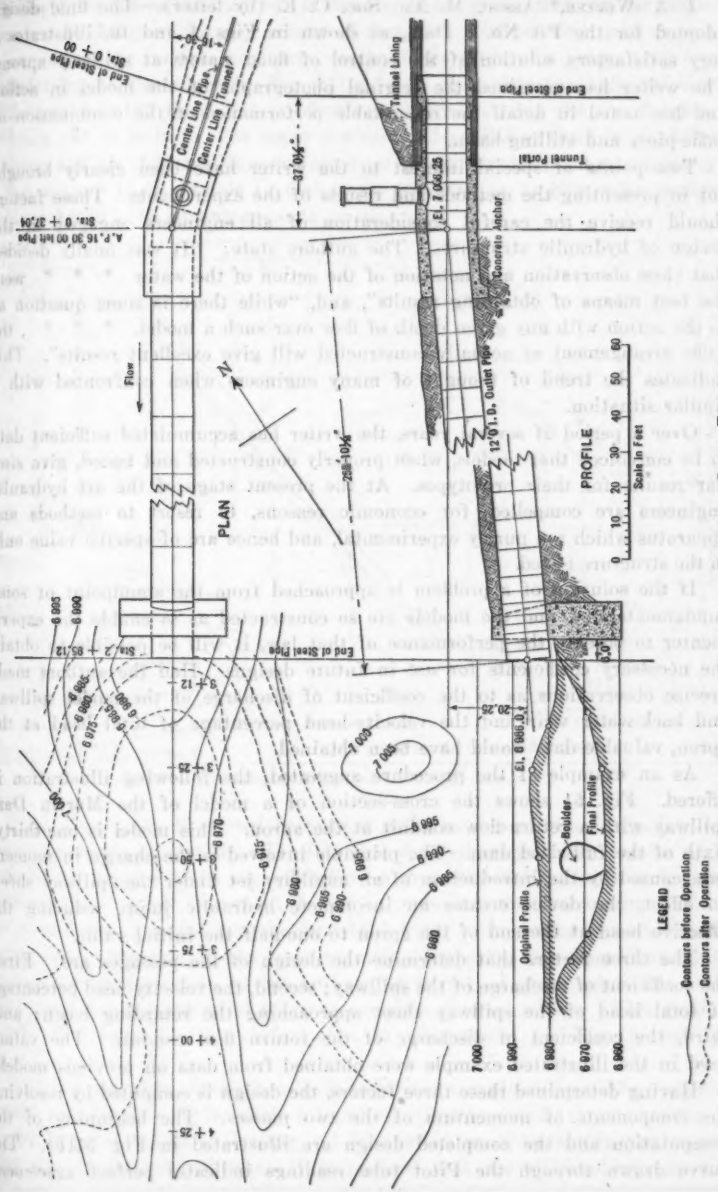


FIG. 50.—PLAN AND PROFILE OF FLORENCE LAKE TUNNEL.

I. A. WINTER,* ASSOC. M. AM. SOC. C. E. (by letter).—The final design adopted for the Pit No. 3 Dam, as shown in Figs. 1 and 10, illustrates a very satisfactory solution of the control of flood waters at spillway aprons. The writer has examined the original photographs of the model in action and has noted in detail the remarkable performance of the combination of baffle-piers and stilling-basin.

Two points of special interest to the writer have been clearly brought out in presenting the methods and results of the experiments. These factors should receive the careful consideration of all engineers engaged in the design of hydraulic structures. The authors state: "It was finally decided that close observation and notation of the action of the water * * * were the best means of obtaining results", and, "while there is some question as to the action with any given depth of flow over such a model, * * * , the baffle arrangement as actually constructed will give excellent results". This indicates the trend of thought of many engineers when confronted with a similar situation.

Over a period of several years, the writer has accumulated sufficient data to be convinced that models, when properly constructed and tested, give similar results for their prototypes. At the present stage of the art hydraulic engineers are compelled, for economic reasons, to resort to methods and apparatus which are purely experimental, and hence are of specific value only to the structure tested.

If the solution of a problem is approached from the standpoint of some fundamental law and the models are so constructed as to enable the experimenter to observe the performance of that law, it will be possible to obtain the necessary coefficients for use in future designs. Had the authors made precise observations as to the coefficient of discharge of the model spillway and back-water weir and the velocity-head percentage of total head at the apron, valuable data would have been obtained.

As an example of the procedure suggested, the following illustration is offered. Fig. 51 shows the cross-section of a model of the Martin Dam spillway with a return-flow conduit at the apron. This model is one-thirty-sixth of the full-sized dam. The principle involved is the change in momentum caused by the introduction of an auxiliary jet under the spillway sheet. In effect, the device creates an incomplete hydraulic jump, reducing the effective head at the end of the apron to one-half the initial value.

The three factors that determine the design of the passages are: First, the coefficient of discharge of the spillway; second, the velocity-head percentage of total head of the spillway sheet approaching the retarding basin; and, third, the coefficient of discharge of the return flow conduit. The values used in the illustrated example were obtained from data on previous models.

Having determined these three factors, the design is computed by resolving the components of momentum of the two masses. The beginning of the computation and the completed design are illustrated in Fig. 51(*b*). The curve drawn through the Pitot tube readings indicates perfect agreement

* Hydr. Engr., Alabama Power Co., Birmingham, Ala.

with the theory as established. It is reasonable to assume that similar structures can be designed for any height of dam, depth of overflow, and change in momentum, up to the maximum depth of the hydraulic jump.

Referring to the curve in Fig. 51(*a*), it is seen that the high velocity at the approach (40 to 55 in.) is effectively reduced during the travel across the basin (55 to 70 in.). The upper surface of the spillway sheet is parabolic in shape and its influence is manifested up stream to the point of tangency of the apron. It is possible to design the basin so as to give a horizontal surface, should it be advantageous to do so.

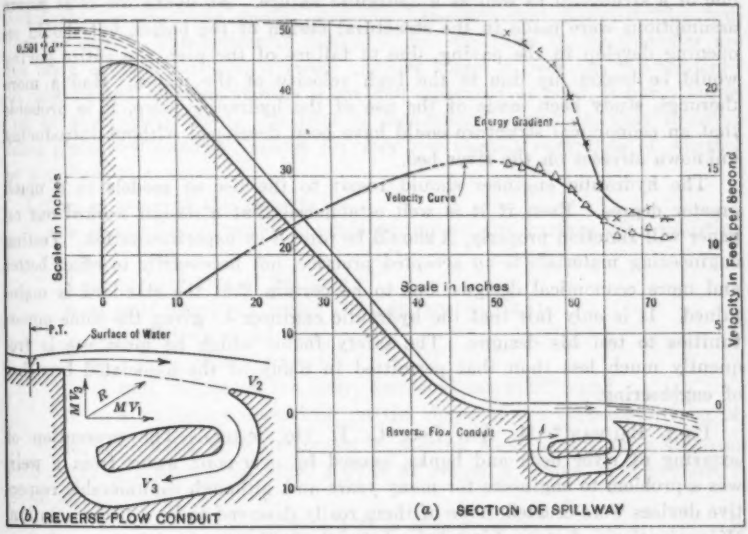


FIG. 51.—MODEL OF MARTIN DAM SPILLWAY.

This illustration is offered because it explains one of the fine points of the design shown in Fig. 51. The fact that the most satisfactory results were obtained by using a 15-ft. step-off indicates that the resulting eddy under the spillway sheet is an important factor in reducing the momentum. Also, the second row of baffles and the final back-water weir creates two additional basins which function in the same manner. Directing the sheet downward, against the baffle-piers is, as the authors point out, of great value, but only when there is sufficient relief below to permit a reversal of flow. This fact is clearly illustrated by the performance in Experiment 11, Fig. 4, and in Table 3.

The use of baffles alone often leads to serious complications. They merely change the direction of the water without effectively reducing the momentum.* If the structure is to be lasting, all dissipation must be within the water. This can be appreciated when it is realized that the stilling-basin at the

* "The Hydraulic Jump as a Means of Dissipating Energy," Technical Rept., Pt. 3, Miami Conservancy Dist.

Pit No. 3 Dam is called upon to dissipate approximately 1 000 000 h.p. If this is not accomplished by the resulting back-water, the hazard to the structure is indeed great.

Concrete is one of the best materials to withstand high velocities at pressures of more than one atmosphere, as long as it is not called upon actually to absorb energy. The writer knows of cases where concrete buckets were used for a similar purpose, with the result that in a very short time the steel reinforcement was exposed.

The question of performance of the design, as shown by the authors, is one of a structural as well as a hydraulic nature. No doubt the most severe assumptions were made in the structural design of the baffles, but should an opening develop in the paving, due to failure of the pier, the entire paving would be broken up due to the high velocity of the water. Had a more thorough study been made of the use of the hydraulic jump, it is probable that an economical structure could have been developed without introducing unknown stresses on the river bed.

The hydraulic engineer should resort to the use of models to a much greater degree. Even if it is well established that a design worked out on paper will function properly, it should be proved by experimentation. Testing engineering materials is an accepted practice, not necessarily to effect better and more economical designs, but to be certain that the standard is maintained. It is only fair that the hydraulic engineer be given the same opportunities to test his designs. The safety factor which he must use is frequently much less than that permitted in many of the associated branches of engineering.

Paul Bauman,* M. Am. Soc. C. E. (by letter).—The prevention of scouring of river beds and banks, caused by over-pour water from a weir, was a problem to engineers for many years and, although innumerable respective devices were created, none of them really deserved to be called a solution. Misconceptions of the subject helped greatly to delay a satisfactory solution, the principal one being the failure to differentiate between the effect of impact of falling water and the effect of a change of the distribution of water-thread velocities as related to their distribution for the normal flow of the river.

Judging from the description of the results of experiments on models of Pit River Diversion Dams Nos. 3 and 4, respectively, it is safe to state that the authors have found a solution, thereby contributing a valuable addition to the knowledge of practical engineers as well as a scientific "nut to be cracked" by those who are interested in the "why" of hydraulic puzzles. Whether or not the arrangement of baffle-piers, as finally chosen by the authors, will prove as successful for an actual flood of 70 000 cu. ft. per sec. as it did for a corresponding discharge over the model dams, is a question which only experience can answer definitely.

The writer, however, believes that, as far as the stilling action is concerned, no appreciable difference between the model and the actual conditions

* Designing Engr., Quinton, Code and Hill, Los Angeles, Calif.

will show unless pulsations should develop due to disturbances in the unprotected river channel below the dam. It must be realized that the natural process of bed-forming in a river is handicapped by any artificial obstruction, such as a weir or a dam, inasmuch as its main factor—the transportation of débris—is more or less interrupted.

Before any hydraulic computation on the weir itself is done, it is, therefore, quite important to study the behavior of the river in its natural bed some distance below. A river flowing on alluvium forms its slope to suit itself. Governing in the formation of the bed is the flow for which the volume of transported débris is a relative maximum. The respective depth is often called the "bed-forming depth" and the respective flow likewise "the bed-forming flow." The hydraulic radius, r, and the slope, s, for this flow are the main characteristics of a river. At low stage, a river has the tendency to raise its bed, whereas at high stage, and especially at flood, it has a lowering or a scouring effect. It must be understood, therefore, that scouring takes place continuously (except for very low stages) regardless of whether or not there is a weir.

Let m = mass of the water;
v = velocity of water threads acting on the surface of a resisting body;
A = surface area of the body;
w = unit weight of water (62.5 lb. per cu. ft.);
μ = coefficient of friction;
V = volume of the water;
γ_0 = unit weight of the body, corrected for uplift; and,
K = a coefficient depending on the shape of the exposed surface, A.

Then, the kinetic force or impact of flowing water acting on a resisting body, such as a particle of sand, a piece of gravel, a pebble, or a boulder, is equal to,

$$F = 1\ P = K\ m\ v \quad \ldots\ldots (1)$$

and, since $m = \dfrac{w\ A\ v}{g}$,

$$P = \frac{K\ w\ A\ v^2}{g} \quad \ldots\ldots (2)$$

and the resistance, R, of the body is equal to,

$$R = \mu\ V\ \gamma_0 \quad \ldots\ldots (3)$$

When $P = R$, motion of the body is impending. Therefore, combining Equations (2) and (3) and solving for v:

$$v = \sqrt{\frac{V\ \gamma_0\ g\ \mu}{K\ A\ w}} \quad \ldots\ldots (4)$$

which is the velocity required to move the body; that is, the water-thread velocity at the bottom of the river. Observations will show, however, that much greater volumes of débris are moved at times than are given by Equation (4). This is due to the fact that, beside the scouring force, P, a force, D, is active as shown in Fig. 52.

Let f = area of cross-section of a water column;

$s = \sin \alpha = \frac{h'}{l}$ = slope of the water surface;

d = depth of the stream; and

μ' = average coefficient of friction.

Then, according to the law of energy,

$$l \times D = w\, d\, f\, h' \quad \ldots\ldots (5)$$

If f is made equal to 1 sq. ft.,

$$D = 62.5\, d\, s \quad \ldots\ldots (6)$$

which is called the dragging force. The resistance, R', of 1 sq. ft. of river bed (see Fig. 53) is,

$$R' = \gamma_0\, t\, \mu' \quad \ldots\ldots (7)$$

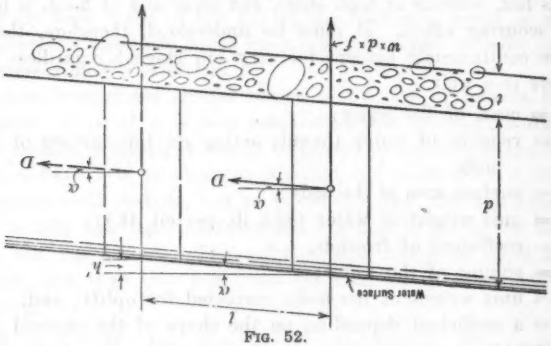


FIG. 52.

and when D is equal to R', Equations (6) and (7) may be combined to determine the depth, t, of the river bed that is in motion, or the limiting size, t of the pebbles or boulders moved. This is equal to,

$$t = \frac{62.5\, d\, s}{\gamma_0 \times \mu'} \quad \ldots\ldots (8)$$

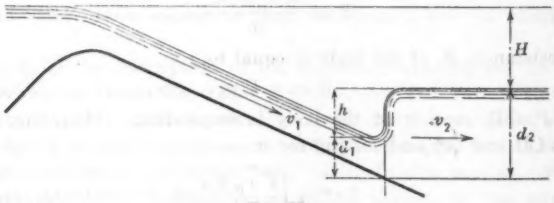


FIG. 53.

As long as t (Fig. 52), therefore, is a little smaller than the smallest diameter of aggregates forming the river bed, no motion will occur. This state of inertia corresponds to a flow during which neither scouring nor settling of débris occurs. It applies particularly to rivers with stony beds, such as the Pit River, because sandy beds are seldom at rest. The abrupt change

in pressure between the water in the upper layers of a sandy river bed and the relatively fast-moving water threads immediately above has a lifting effect on the top particles which, once loosened, are readily carried away. The vertical velocity corresponding to this change in pressure has been observed to be from one-tenth to one-thirtieth of the horizontal velocity.

It is quite important to design the weir with due respect to the natural conditions of the river. The main source of scouring of over-pour water only too often lies in a poor adjustment of the weir to the natural bed of the river and above all of the apron. Weir construction is done as often as possible at low water and the temptation to put the apron on top of the then existing bed with a minimum of excavation is considerable. The bed-forming flow alone sometimes will lower the bed enough to cause the water to jump off the high apron, producing impact and eddies and probably doing more damage than a great flood would with a well-designed apron. The aforementioned handicap to the shifting of débris adds to the seriousness of this occurrence.

If the weir is assumed to be built so as to prevent percolation, it is governed by hydrodynamic, static, and, last but not least, economic features only. As to the first, it must be remembered that all the mathematically derived equations of hydraulics must be supplemented with empirical coefficients in order to make them applicable for practical use. Although these coefficients may often be chosen successfully from those determined for similar conditions elsewhere, the most reliable determination, undoubtedly, will result from experiments on models representing the actual conditions as closely as possible. This has been done by the authors. The greater the velocities involved in the problem, the greater the discrepancy between the strictly theoretical and the actually observed conditions and, therefore, the greater will be the importance of experiments because little is known about the "rushing or shooting" flow; that is, the flow at an average velocity, v, greater than wave velocity, v_w, inasmuch as ordinary hydraulic formulas strictly apply to "quiet" flow; that is, for $v < v_w$ only.

The discharge from a weir usually represents the case where water of higher velocity is absorbed by water of lower velocity. If the change of velocity would occur very suddenly or, in other words, along a cross-sectional plane and the absolute velocity of the tail-water were rather small, a hydraulic jump would occur, mathematically expressed as follows (Fig. 53):

$$v_1 \, d_1 \, b = v_2 \, d_2 \, b \ldots\ldots\ldots\

By algebra,

$$d_2^2 - d_1^2 = (d_2 + d_1)(d_2 - d_1) \quad \ldots\ldots(13)$$

Substituting Equation (13) into Equation (12) and solving for:

$$d_2 = \frac{v_1^2}{4g} \pm \sqrt{\frac{v_1^2}{2g}\left(\frac{v_1^2}{8g} + d_1\right)} \quad \ldots\ldots(14)$$

which is a well-known formula. Note that, since d_2 cannot become negative, the sign before the radical is always plus. If the discharge is altered so as to diminish the height, h, of the jump until it finally becomes zero, then $d_2 = d_1$. Substituting d_1 for d_2 in Equation (14) and solving for v_1:

$$v_1 = \sqrt{g\, d_1} = v_w \quad \ldots\ldots(15)$$

or,

$$d_1 = \frac{v_1^2}{g} = 2\, h_1$$

This means that in order to produce a jump, the velocity head, h_1, must be at least as great as one-half the depth, or the velocity, v_1, must be at least equal to the wave velocity, v_w. Another condition for the elimination of a jump follows from,

$$h = \frac{v_1^2 - v_2^2}{2g} \quad \ldots\ldots(16)$$

in which, h becomes zero for $v_1 = v_2$ or, in other words, no jump occurs if the tail-water velocity is equal to or greater than the velocity of the over-pour water in the plane of contact. Neither the latter condition nor the one for a hydraulic jump, however, is ordinarily present where a flood discharge over a weir in a (natural) river is concerned, but one or more standing waves may form as an "outburst" of kinetic energy accumulated in the falling water, due to a more or less sudden change in slope. While, in principle, the standing wave is nothing but a modified jump, its height is not determinable by means of Equation (14) for weirs with a relatively high drop, H (Fig. 54), and only approximately, for those with a relatively low drop. The reason is that only part of the kinetic energy is transformed into potential energy while the remainder is transformed into thermal energy due to the formation of a superficial eddy. Inasmuch as this energy is non-injurious to either the protected or the unprotected river bed, it is evident that the tendency must be to decrease the undesirable standing wave and to encourage the formation of a large superficial eddy. In principle, this may be accomplished by means of a smooth transition between the down-stream slope of the weir and the apron (Fig. 54) and by the proper width, length, and elevation of the latter or, in other words, by providing a sufficient depth, d_2. The efficiency of this eddy depends on the ratio of $\frac{H}{d_2}$ (Fig. 54), which is best determined on models.

The absorption of kinetic energy takes place to such a degree that the water emerging from beneath the superficial eddy always flows with an average velocity that does not exceed the wave velocity. The volume, in cubic feet,

of a superficial eddy required to perform this transformation of energy has been determined by Professor Theodor Rehbock* as being between,

$$V_E = 3.60\ Q \sqrt{\frac{H}{g}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (17)$$

and,

$$V_E = 7.20\ Q \sqrt{\frac{H}{g}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (18)$$

in which, Q equals discharge, in cubic feet per second, and H equals drop (see Fig. 54).

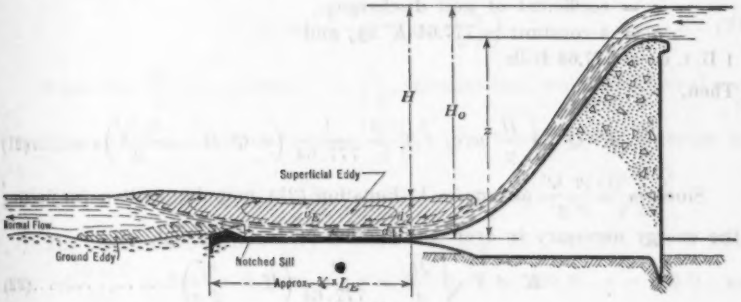


FIG. 54.

The two limits of Equation (17) indicate that an efficiency is involved which, considering the fact that the superficial eddy is transforming energy, is quite plausible.

If K = a constant, corresponding to 3.60 in Equation (17);
Q' = discharge per unit width of channel;
η = efficiency in percentage; and
H_0 and d_2 are as shown in Fig. 54;
Equation (17) may be written,

$$V_E' = \frac{K}{\eta}\ Q' \sqrt{\frac{H_0 - d_2}{g}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (19)$$

The efficiency is then given by:

$$\eta = \frac{K\ Q'}{V_E'} \sqrt{\frac{H_0 - d_2}{g}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (20)$$

It is evident that the efficiency, η, can be determined on models by computing the numerator, assuming that Q' and H_0 are constant and that d_2 is variable, and by measuring the respective cross-sectional area, V_E'. Plotting the resulting values of η as ordinates and the corresponding values of d_2 as abscissas will give points on a curve such that the maximum indicates the desirable depth, d_2, which should be proportionally applied to the weir as closely as conditions permit.

* "Betrachtungen über Abfluss, Stau- und Walzenbildung bei fliessenden Gewässern", Berlin, 1917, Jul. Springer.

Let $K' = \frac{K}{\eta}$, which is constant for a small change of H and v_2;

c_p = specific heat of water;

ΔT = increase of temperature within the superficial eddy;

L_E = effective length of the superficial eddy;

Q'' = discharge per linear foot of weir $= \frac{B}{b} Q'$;

b = length of weir crest;

B = width of apron;

z = height of weir crest above apron;

c = coefficient of weir discharge;

a = a constant $= 777.64\ K'\ 2g$; and

1 B. t. u. = 777.64 ft-lb.

Then,

$$K'\ Q' \sqrt{\frac{H}{g}}\ w\ c_p\ \Delta T = \frac{1}{777.64} \left(w\ Q'\ H - \frac{m\ v_2^2}{2} \right) \ldots\ldots\ldots (21)$$

Since $m = \frac{w\ Q'}{g}$ and $c_p = 1$, Equation (21) may be written (neglecting the energy necessary to overcome friction on the way, L_E),

$$K'\ \Delta T \sqrt{\frac{H}{g}} = \frac{1}{777.64} \left(H - \frac{v_2^2}{2\ g} \right) \ldots\ldots\ldots\ldots (22)$$

and,

$$\frac{777.64\ K'\ 2\ g}{\sqrt{g}}\ \Delta T = 2\ g \sqrt{H} - \frac{v_2^2}{\sqrt{H}} \ldots\ldots\ldots (23)$$

Substituting the value, a:

$$\Delta T = \frac{1}{a} \left(2\ g \sqrt{H} - \frac{v_2^2}{\sqrt{H}} \right) \ldots\ldots\ldots\ldots (24)$$

Considering that $v_2 = \frac{Q'}{d_2}$, and $Q' = \frac{b}{B} Q''$, Equation (21) can be solved into the form:

$$\Delta T = \frac{2\ g}{a} \left(\sqrt{H} - \frac{b^2\ c^2\ (H + d_2 - z)^3}{B^2\ d_2^2\ \sqrt{H}} \right) \ldots\ldots\ldots (25)$$

The total derivation of ΔT by the parameters, H and d_2, is given by:

$$\delta\ (\Delta T) = \frac{\delta\ (\Delta T)}{\delta\ H}\ d\ H + \frac{\delta\ (\Delta T)}{\delta\ d_2}\ d\ (d_2) \ldots\ldots\ldots (26)$$

and the superficial eddy is most effective if ΔT becomes a maximum, which means that $\frac{\delta\ (\Delta T)}{\delta\ H} = 0$ and $\frac{\delta\ (\Delta T)}{\delta\ (d_2)} = 0$, simultaneously, and,

$$\frac{\delta\ (\Delta T)}{\delta\ (H)} = 0 = \frac{1}{\sqrt{H}}$$

$$d_2^2 \sqrt{H}\ B^2\ 3\ c^2\ (H + d_2 - z)^2\ 1\ b^2 - c^2\ (H + d_2 - z)^3\ b^2\ \frac{d_2^2}{\sqrt{H}}\ B^2$$

$$\overline{\qquad\qquad\qquad\qquad\qquad\qquad\qquad\qquad\qquad\qquad\qquad\qquad} \ldots (27)$$

$$d_2^4\ H\ B^4$$

$$0 = d_2^2\, B^2 - b^2\, 3\, c^2\, (H + d_2 - z)^2 + b^2\, c^2\, (H + d_2 - z)^3\, \frac{1}{H} \ldots\ldots (28)$$

$$\frac{\delta\,(\Delta\, T)}{\delta\,(d_2)} = 0$$

$$= - \frac{d_2^2 \sqrt{H}\, B^2\, 3\, c^2 (H + d_2 - z)^2\, 1\, b^2 - c^2 (H + d_2 - z)^3\, b^2\, 2\, d_2 \sqrt{H}\, B^2}{d_2^4\, H\, B^4} \ldots\ldots (29)$$

$$0 = - d_2\, 3\, c^2\, (H + d_2 - z)^2\, b^2 + 2\, c^2\, (H + d_2 - z)^3\, b^2 \ldots\ldots\ldots (30)$$

By adding Equations (28) and (30), substituting $\frac{(Q'')^2}{2\,g}$ for $c^2\,(H + d_2 - z)^3$, and simplifying the equation, the result is:

$$d_2^3 = \frac{(Q'')^2\, b^2}{g\, B^2} \left(1 - \frac{d_2}{2\,H}\right) \ldots\ldots\ldots\ldots\ldots\ldots (31)$$

When $H = \frac{d_2}{2}$, the quantity, $1 - \frac{d_2}{2\,H}$, becomes zero, which indicates at once that it is a limiting condition because $\frac{(Q'')^2\, b^2}{g\, B^2}$ then becomes $+\ \infty$, whereas, for $H = +\ \infty$, the quantity, $1 - \frac{d_2}{2\,H}$, becomes $+\ 1$ and $d_2 = \sqrt[3]{\frac{(Q'')^2\, b^2}{g\, B^2}}$.

The result of Equation (31) is shown in Fig. 55. At Pit No. 3 Dam, $Q'' = \frac{70\,000}{267} = 262$ cu. ft. per sec.; $b = 267$ ft.; $B = 200$ ft., $z = 102$ ft.; and $H_0 = 120$ ft. Therefore, $H = 104.75$ ft. and d_2 (by Equation (31)) $= 15.25$ ft. At Pit No. 4 Dam, $Q'' = \frac{70\,000}{136} = 515$ cu. ft. per sec.; $b = 136$ ft., $B = 160$ ft., $z = 40$ ft. $\pm$; and $H_0 = 67$ ft. $\pm$. Under these conditions, $H = 50.00$ ft. and $d_2 = 17.00$ ft.

As to the actual conditions, it is estimated that for $Q = 70\,000$ cu. ft. per sec., $d_2 = 17.00$ ft., and $H = 103.00$ ft., at Pit No. 3 Dam, and $d_2 = 21.00$ ft., and $H = 46.00$ ft., at Pit No. 4 Dam, which shows enough similarity to the depth, d_2, and the drop, H (as given by Fig. 55), to warrant a very effective superficial eddy. In computing its volume, therefore, it seems permissible to use,

$$V_E = 3.60\ Q \sqrt{\frac{H}{g}}$$

For Pit No. 3 Dam,

$$V_E = 3.60 \times 70\,000 \sqrt{\frac{103}{32.20}} = 452\,000 \text{ cu. ft.}$$

or a cross-sectional area of,

$$A_E = \frac{452\,000}{200} = 2\,260 \text{ sq. ft.}$$

The mean depth, $d_E = d_2 - d_1$ (Fig. 54). As (Fig. 56) d_1 is, roughly, 4 ft., $d_E = 17 - 4 = 13$ ft., which gives an effective length,

$$L_E = \frac{A_E}{d_E} = \frac{2\,260}{13} = 175 \text{ ft., more or less.}$$

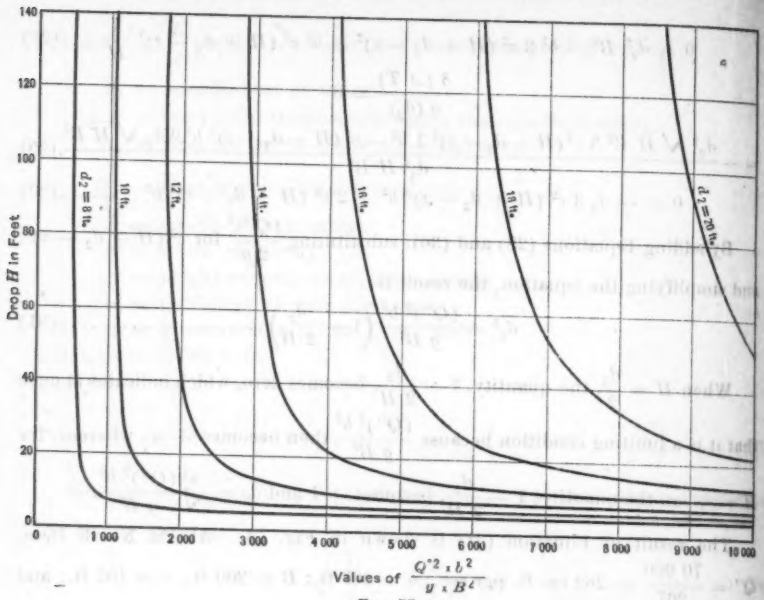

Drop H in Feet
Values of $\frac{Q^2 \times b^2}{g \times B^2}$
$d_2 = 8$ ft.
10 ft.
12 ft.
14 ft.
16 ft.
18 ft.
$d_2 = 20$ ft.

FIG. 55.

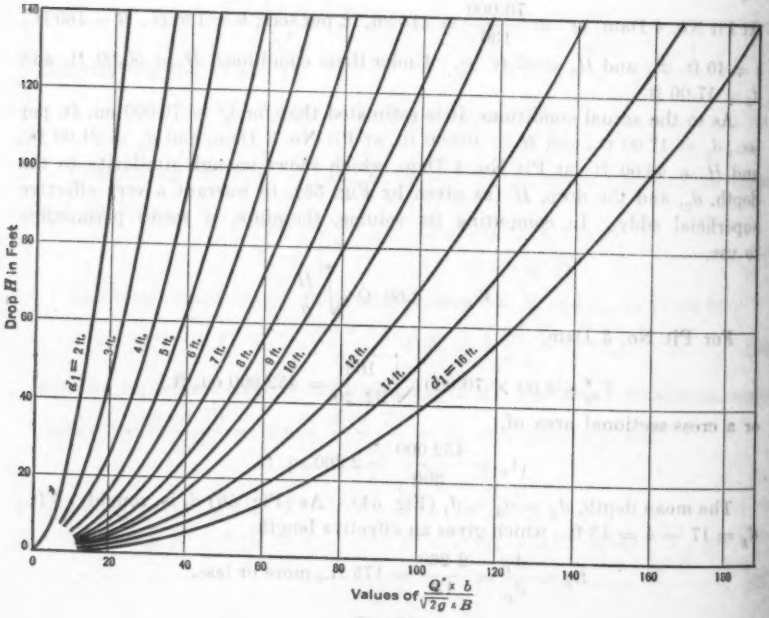

Drop H in Feet
Values of $\frac{Q \times b}{\sqrt{2g} \times B}$
$d_1 = 2$ ft.
3 ft.
4 ft.
5 ft.
6 ft.
7 ft.
8 ft.
9 ft.
10 ft.
12 ft.
14 ft.
$d_1 = 16$ ft.

FIG. 56.

Similarly, for Pit No. 4 Dam,

$$V_E = 3.60 \times 70\,000 \sqrt{\frac{46}{32.20}} = 300\,000 \text{ cu. ft.}$$

$$A_E = \frac{300\,000}{160} = 1\,875 \text{ sq. ft.}$$

$$d_E = d_2 - d_1 = 21 - 8 = 13 \text{ ft.}$$

and,

$$L_E = \frac{1\,875}{13} = 145 \text{ ft., more or less.}$$

From this it is evident that baffle-piers or any other kind of obstructions are superfluous in all cases where the river bed is protected by an apron under the full length of the superficial eddy as far as impact and kinetic energy are concerned. If, in spite of the effect of absorption, dangerous scouring occurs for a considerable distance down the river, it is due to the fact that the highest velocities of the emerging water occur at the bottom because of the absorption of energy from the surface. This is contrary to the normal flow conditions in a river where the velocity immediately above the bottom is a relative minimum. To prevent scouring, due to these high velocities (see Equation (4)), at the bottom, the river bed would have to be protected far beyond the lower end of the superficial eddy which, however, in most cases, would add to the cost in such a way as to make the structure economically impossible.

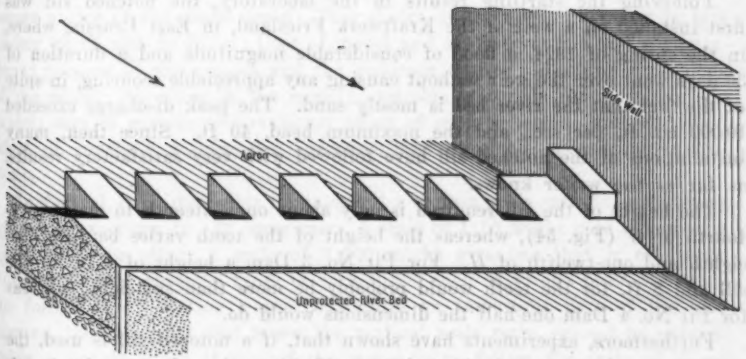


FIG. 57.—NOTCHED SILL.

Guided by these considerations Professor Rehbock has invented a device called "Zahnschwelle",* or "Notched Sill" (Fig. 57), which is placed at the down-stream end of the apron for the sole purpose of establishing the natural flow conditions immediately below. As the average velocity at the down-stream end of the apron is smaller than the wave velocity, the kinetic energy and its action on the notched sill is small as compared with that on the baffle-piers, which are exposed to the most terrific onslaught of falling water,

* U. S. Patent No. 1 561 796.

and, therefore, call for the best reinforced concrete, strengthening of the apron, and, last but not least, sound foundations. It must be realized that at Pit No. 3 Dam, for instance, the better part of about 800 000 kinetic h.p. is "killed" within the zone of the baffle-piers. This undoubtedly calls for adequate structures, especially where concave surfaces are struck, lest the "killing" is done by the "horses" instead of the piers.

It is for these reasons and the fact that the notched sill has successfully stood the actual flood tests that the writer ventures to call it the best solution of the scouring problem, both from a scientific and an economic standpoint.

The principle of its action is that the rushing water sheets immediately above the apron are deflected in an upward direction by the vertical face of the teeth, thereby producing a long and flat ground eddy immediately below the sill which revolves as shown in Fig. 54; that is, its lowest sheets move in an up-stream direction. This movement is counteracted by the lower threads of fan-shaped jets pressed through the gaps between the teeth so as mutually to "destroy" energy without appreciable scouring of the unprotected river bed (deposition has been observed in most cases instead). The upper fan-shaped threads prevent the main stream, deflected upward by the teeth, from dropping down on the bottom which it would otherwise tend to do. Through the concentration of water threads near the surface, the discharge velocities there are increased. This, with the simultaneous decrease of the bottom velocities, leads to the normal distribution of velocities in the discharging stream.

Following the startling results in the laboratory, the notched sill was first installed on a weir of the Kraftwerk Friesland, in East Prussia, where, in the spring of 1924, a flood of considerable magnitude and a duration of 30 days went over the weir without causing any appreciable scouring, in spite of the fact that the river bed is mostly sand. The peak discharge exceeded 10 000 cu. ft. per sec., and the maximum head, 40 ft. Since then, many installations of the notched sill have followed with very satisfactory results, as far as the writer knows.

The height of the sill required is only about one-sixteenth to one-twenty-fourth of H (Fig. 54), whereas the height of the teeth varies between one-eighth and one-twelfth of H. For Pit No. 3 Dam a height of 4 ft. for the sill and 8 ft. for the teeth would probably be more than sufficient; whereas for Pit No. 4 Dam one-half the dimensions would do.

Furthermore, experiments have shown that, if a notched sill is used, the apron may advantageously be only two-thirds to three-fourths the length of the superficial eddy, or, in the present case, say, 120 to 130 ft. long for Pit No. 3 Dam and 100 to 116 ft. long for Pit No. 4 Dam, respectively.

The writer is indebted to the authors for their kind co-operation in furnishing plans necessary for the calculations involved.

MORROUGH P. O'BRIEN,* JUN. AM. SOC. C. E. (by letter).—Although a few model experiments have been made in the United States, the prevailing opinion among American hydraulic engineers seems to have been, at least

* Asst. Prof., Mech. Eng., Univ. of California, Berkeley, Calif.

until very recently, that they are more a matter of academic interest than a basis for actual design. The present series of experiments are encouraging since they indicate that this attitude is changing. A recent German publication, "Die Wasserbaulaboratorien Europas", which describes the European hydraulic laboratories and their methods of study, shows that in those countries where engineering knowledge is most advanced and economic pressure great, model experiments find their widest use as a guide to hydraulic design. In the introduction to this volume, Dr. de Thierry, one of the foremost hydraulic engineers in Germany, makes the statement that, far from being an expensive luxury, laboratories for model studies are an absolute necessity for the economic design of hydraulic structures.

In the use of models for hydraulic studies, there are certain limitations which, if they are not known and properly taken into account, may cause the results to be grossly in error. The principles of dimensional analysis and dynamic similarity, on which the whole theory of models is based, cannot be applied blindly. In deriving these model laws, certain assumptions are made and, consequently, the conditions in the models must be carefully analyzed to see whether these assumptions are approximately realized. From an examination of many model experiments, the writer is of the opinion that a scale effect is to be expected in nearly every type of model, but that, by carefully regulating the conditions in the model, an experienced investigator can keep this scale effect or error within allowable limits.

In making model studies, the first step, and the most important one, is the proper choice of the model scale. This choice is dependent on many factors among which may be listed: Cost of model and of observations and computations; purpose of experiment and the accuracy desired; absolute dimensions of the model; velocities in the model; available materials for surfacing the model; ease of making observations; effect of surface tension and viscosity; effect of constant atmospheric pressure; and available measuring instruments.

In general, it is impossible to state that a certain scale ratio is satisfactory for model experiments. In some cases, a scale of 1:10 might give too small a model while in others a scale of 1:500 might give one that is too large. For experiments on some types of structures, such as bridge piers, in which the formation of surface waves constitutes an important part of the phenomena, the velocities, and hence the vertical scale ratio, cannot be reduced below a certain point if size of the waves is to follow the scale of linear quantities. If there is a possibility of a reduction in pressure at important points in the model, the scale of the model is limited by the fact that the atmospheric pressure is the same for both model and prototype. In river models, where the depths and velocities are likely to be small, the manner of flow may be determined by the viscosity. The scale ratio chosen should be such that the cost of the experiment will not be excessive and that the results will be sufficiently reliable for the purpose at hand. Besides using geometrically similar models, it is possible to reduce the cost by using distorted models, vertical sections, and half-models. The studies of the Berak Power

Plant, in India, were made with a complete model to a scale of 1:96 and with vertical sections through the sector gates, bottom outlets, and spillways to scales of 1:36 and 1:52. The hydraulic and economic requirements are always in opposition and the proper compromise between them can only be made after considerable experience in this kind of work.

The accuracy and reliability of experiments on models depend on the scale ratio and, of course, on the accuracy of observation. Studies at the Hydraulic Research Institute, at Munich, Germany, show a very good agreement, both as to the discharge and the erosion, between a model built to a scale of 1:50 and its prototype, one of the small dams of the Mittlere Isar Project. In these experiments the size of the model was reduced by making it a half-model which was allowable since the structure was symmetrical about the center line of the stream. A few years ago a model of the Harbor of Stockholm, Sweden, constructed to a horizontal scale of 1:300 and a vertical scale of 1:100, was tested at the Technical University of Stockholm, and it was found that the velocities indicated by the model agreed within 10% with those in the harbor. This divergence was decreased when the layer of slightly saline sea water in the harbor was reproduced in the model to the proper scale. While the turbines at Lille Edet were being tested, a rough check was made on the agreement between the actual conditions of flow and those shown by the model, which had been used in the design. It was only possible to examine the principal currents, but these were found to agree exactly.

The great savings in cost of construction that are very often made possible by model experiments are shown very clearly in the publication previously mentioned describing the European laboratories. As an additional instance may be cited the power plant of the Swedish Government at Trollhättan where it was proposed to increase the head on the plant by constructing underground conduits from the tail-race to a point in the river below a controlling section. The proposed plan would have cost about 3 000 000 Swedish crowns. A model experiment was made at the Technical University and it was found that these conduits would not have the desired effect, due to peculiar eddies at the intake. As a result of the model studies it was decided to place much shorter conduits on the opposite bank of the river at a cost of 900 000 crowns. Full-scale experimentation would have wasted several million crowns.

In making experiments on river models, particularly on the dissipation of kinetic energy at the foot of a spillway, it is of the greatest importance to know the water level in the river bed below the dam for different stages of flow. If a controlling section occurs below the apron, it is necessary to increase the roughness of the model locally until the proper flow line is obtained. Otherwise, the depth of flow can usually be controlled by means of an adjustable weir.

J. C. STEVENS,* M. AM. SOC. C. E. (by letter).—In November, 1927, the writer conducted some experiments on a one-twelfth size model of the diversion dam for the Leaburg hydro-electric development. This is the

* Cons. Hydr. Engr. (Stevens & Koon), Portland, Ore.

second of a series of hydro-electric power plants to be constructed on McKenzie River by the City of Eugene, Ore.

The object of the experiments was to ascertain (1) the effect the proposed dam would have on flood heights above it; (2) the type of baffles on the apron most effective in preventing scour; (3) the most satisfactory relation between height of dams and height of gates; and (4) the manner in which the flood-gates should be operated, with the apron and baffles selected to avoid scouring the river bed. This development consists essentially of a diversion dam to raise the river level 20 ft., a canal of 2 200 sec-ft. capacity, 5 miles in length, and a power house where this flow is utilized under 90 ft. of head, thereby creating 20 000 h. p.

The Leaburg Diversion Dam will consist of a low overflow weir with roller type of gates on the crest, 100 ft. long and 12½ ft. high, supported by piers. There are three such gates and, in addition, a 30 by 18-ft. sluice-gate, for auxiliary regulation.

Provision is made for passing 90 000 sec-ft. over the dam with all gates open. The dam must not raise flood water more than 2½ to 3 ft. higher than would occur in the river without it. A field laboratory was constructed to which water could be drawn under control from the Walterville Canal, which supplies City Plant No. 1.

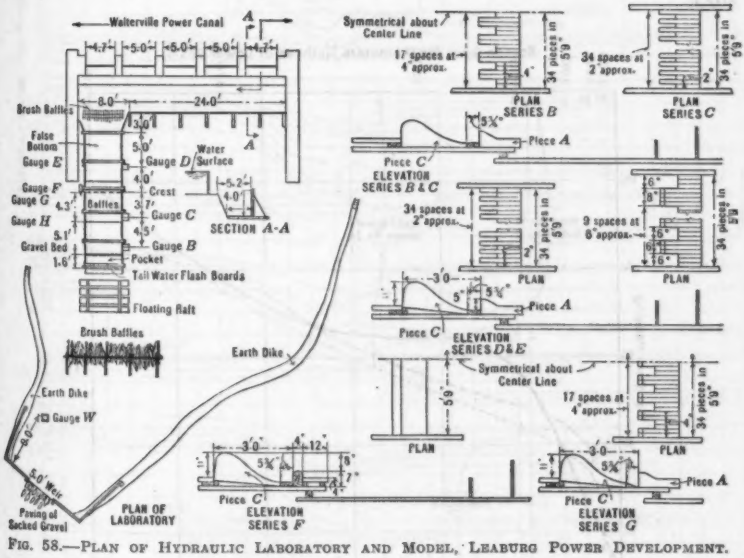


FIG. 58.—PLAN OF HYDRAULIC LABORATORY AND MODEL, LEABURG POWER DEVELOPMENT.

Fig. 58 shows a plan of the laboratory and the dam and baffles used for the six series of experiments made. Fig. 59 shows the setting of the model and baffles in the flume. It was specified that stock for Pieces *A*, *B*, and *C* be cut to the stock sizes shown and then soaked in water completely

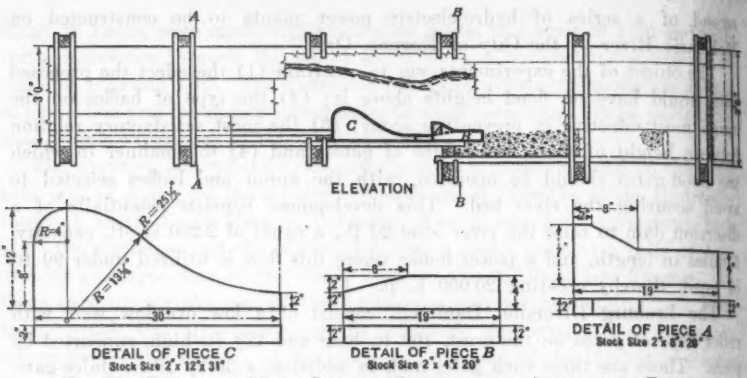


FIG. 59.—DETAILS OF MODEL DAM AND BAFFLES, WITH ASSEMBLY IN FLUME, LEABURG POWER DEVELOPMENT.

submerged for at least ten days prior to milling. After milling, the pieces were submerged in water until used. The object was to secure pieces accurately dimensioned when thoroughly saturated. It was permitted to use No. 1 clear white pine, cedar, or redwood, but all must be of the same stock.

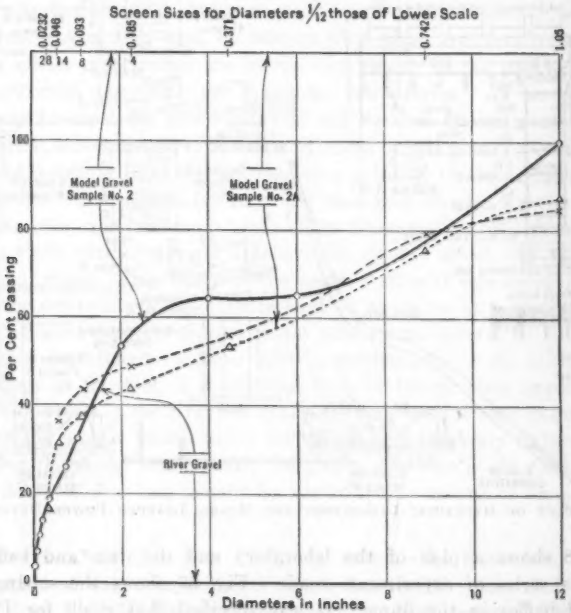


FIG. 60.—GRADING OF RIVER AND MODEL GRAVELS.

Water was drawn from the wasteways of the power canal into a 6 by 4-ft. timber flume. It was quieted above the dam by brush baffles; then it was passed over the model dam and gravel bed, dropped into a small pond, and thence, through another set of brush baffles and over the 5-ft. rectangular measuring weir.

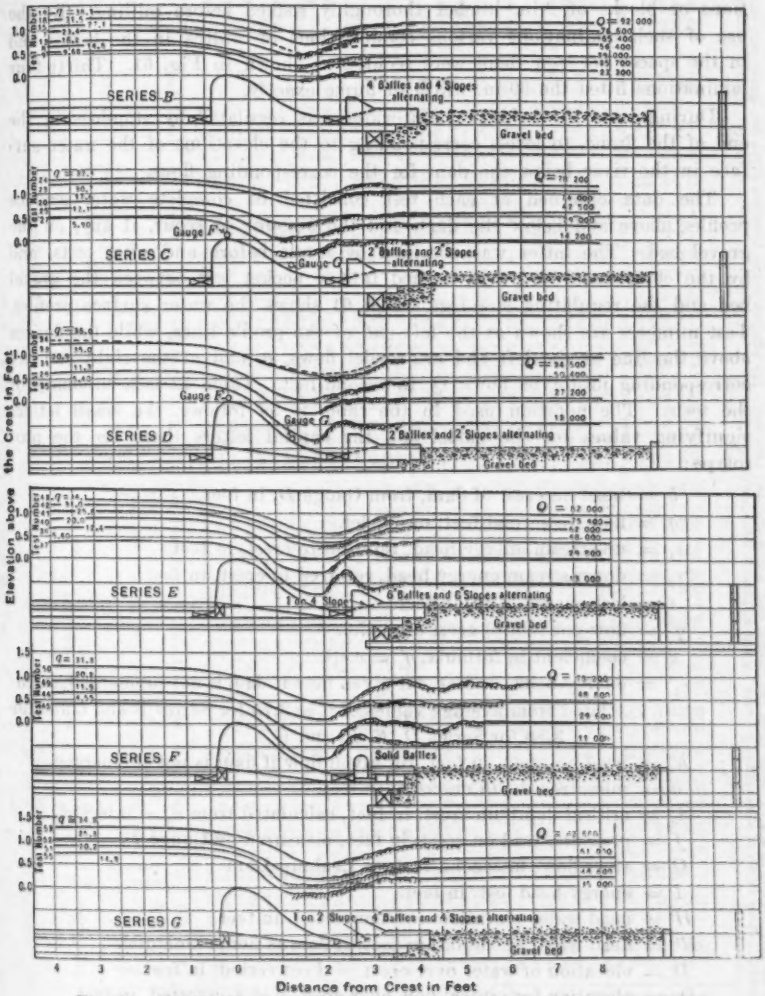


FIG. 61.—EXPERIMENTS ON MODEL DAM, LEABURG POWER DEVELOPMENT.

After passing the aprcn with its baffles, the water flowed over a bed composed of sand and small gravel graded similarly, but sized as nearly as possible to one-twelfth those of the bed of the prototype. How nearly

the prototype gravel bed was imitated in the model is shown in Fig. 60. The river gravel tested was a composite of five 100-lb. samples taken from the river bed at the dam site. The model gravels were made up from screened and recombined river sands and gravels.

As shown on Fig. 59 the model dam and baffles were made up of laminations or blocks of 2-in. lumber thoroughly soaked before milling. By the use of such laminations various combinations of widths in the baffles and in the spaces between them were secured as shown in Fig. 61. Thirty-four laminations fitted the 69-in. width of flume exactly.

During tests the height of tail-water was regulated by stop-logs in the end of the flume, to levels corresponding to the elevations of the water surface in the river below the dam for the corresponding flows.

The data obtained at each test consisted of complete water surface profiles above and below the dam, and the amount of scour, if any, on the gravel bed. The latter was obtained by levels before and after tests, and by the observance of gravel washed into a pocket left between the gravel bed and the regulating stop-logs. Fig. 61 shows the water surface profiles. Test numbers are shown at the left end of the profile lines, while the figures above the line at the left end are model flows, q, and at the right end are corresponding prototype flows, Q, in second-feet. Table 12 is a summary of the tests. The notation used in the table is as follows, the small letters signifying values for the model and the capital letters those for the prototype:

h = head on crest of dam, from Gauge D, in feet.
q = flow, in second-feet, on model.
e_1 = up-stream energy head, referred to crest, in feet.
e_2 = down-stream energy head, referred to crest, in feet.
i = lost energy head, in feet.
q' = flow per foot of crest of model.
c = coefficient in formula, $q' = c\, e_1^{\frac{3}{2}}$.
e_1' = energy head, in feet, for given flow if dam is not submerged, calculated from average values of $c = 3.75$ for Series B and C and for $c = 3.80$ for Series D, E, F, and G.
h' = head on crest, in feet, for given flow if dam is not submerged.
d = measured depth on crest, in feet.
d_c = critical depth on crest, in feet, calculated from $d_c = 0.098\, q^{\frac{2}{3}}$.
f = pressure head on crest, in feet, measured by Gauge F.
Q = river flow, in second-feet, at Leaburg Dam.
I = energy head lost, in feet.
H = head water elevation = h converted, in feet.
H' = head-water elevation, in feet, if dam is not submerged.
D = elevation of water over crest = d converted, in feet.
D_c = elevation for critical flow over crest = d_c converted, in feet.
F = elevation of pressure head on crest = f converted, in feet.

Fig. 62 shows various hydraulic curves pertaining to the experiments, all referred to the prototype.

TABLE 12.—SUMMARY OF EXPERIMENTS ON MODEL OF LEABURG DIVERSION DAM AND CONVERSION TO PROTOTYPE QUANTITIES.

Series and test.	On Model.														On Prototype.									Remarks.
	Head, h, in feet.	Flow, q, in second-feet.	Tail-Water.		Submergence, percentage.	Energy Loss.			For Unsubmerged Flow.		On Crest.				Head, H, in feet.	Flow, Q, in second-feet.	Tail-Water.		l, in feet.	H', in feet.	D, in feet.	D_c, in feet.	F, in feet.	
			Actual, in feet.	Normal, in feet.		e_1, in feet.	e_2, in feet.	i, in feet.	q', in second-feet.	c.	e_1', in feet.	k', in feet.	d, in feet.	d_c, in feet.	f, in feet.		Actual, in feet.	Normal, in feet.						
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)	(23)	(24)	(25)	(26)
B- 7	0.97	23.4	0.79	0.73	82	1.06	0.90	0.16	4.06	.....	1.05	0.97	0.85	0.80	....	739.1	56 400	737.0	737.0	1.92	739.1	737.7	737.1	.....	No scour of gravel bed.
8	0.69	14.8	0.37	0.48	54	0.74	0.45	0.29	2.56	.....	0.77	0.72	0.58	0.59	....	735.8	35 700	731.9	732.7	3.48	736.1	734.4	734.6	.....	
9	0.56	9.68	0.15	0.18	27	0.59	0.21	0.38	1.68	3.74	0.59	0.56	0.44	0.44	....	734.2	23 300	729.3	729.6	4.56	734.1	732.8	732.8	.....	
12	0.76	16.2	0.51	0.48	67	0.82	0.59	0.23	2.81	3.80	0.83	0.77	0.63	0.63	....	736.6	39 000	733.6	733.4	2.76	736.7	735.1	735.1	.....	
15	1.10	26.9	0.93	0.94	84	1.21	1.06	0.15	4.68	.....	1.16	1.05	1.00	0.88	....	740.7	64 900	738.7	738.7	1.80	740.1	739.5	738.1	.....	Some fines washed into pocket.*
16	1.02	26.4	0.80	0.92	78	1.13	0.95	0.18	4.60	.....	1.15	1.04	....	0.87	....	739.7	63 600	737.1	738.5	2.16	739.9	.....	737.9	.....	
17	1.00	25.6	0.68	0.89	68	1.11	0.83	0.28	4.45	3.80	1.12	1.01	....	0.85	....	739.5	61 600	735.7	738.1	3.86	739.6	.....	737.7	.....	
18	1.23	31.5	1.07	1.10	87	1.36	1.24	0.12	5.48	.....	1.27	1.13	1.14	0.98	....	742.2	76 500	740.3	740.7	1.44	741.0	741.2	739.3	.....	Slight scour.
19	1.37	38.1	1.28	1.32	90	1.53	1.41	0.12	6.80	.....	1.49	1.31	1.29	1.11	....	743.9	92 000	742.3	743.4	1.44	743.2	743.0	740.8	.....	Considerable scour.
C-20	0.79	17.6	0.57	0.56	72	0.85	0.66	0.19	3.06	.....	0.87	0.80	0.67	0.66	....	737.0	42 500	734.3	734.1	2.28	737.1	735.5	735.4	.....	No scour.
22	1.17	31.2	0.96	1.09	82	1.30	1.18	0.14	5.48	.....	1.28	1.14	....	0.97	....	741.5	75 000	739.0	740.4	1.68	741.1	.....	739.1	.....	Some fines washed into pocket.
23	1.21	30.6	1.06	1.07	88	1.33	1.20	0.13	5.33	.....	1.26	1.12	1.12	0.96	....	742.0	73 700	740.2	740.2	1.56	740.9	740.9	739.0	.....	
24	1.29	32.3	1.17	1.13	90	1.41	1.31	0.10	5.62	.....	1.31	1.17	1.21	0.99	....	743.0	77 500	741.5	741.0	1.20	741.5	742.0	739.4	.....	
25	0.65	12.1	0.28	0.30	43	0.69	0.35	0.34	2.10	3.70	0.68	0.64	0.50	0.52	0.39	735.3	29 000	730.9	731.1	4.09	735.2	733.5	733.7	732.2	No scour.
27	0.42	5.90	—0.06	—0.06	0	0.43	—0.02	0.45	1.02	(3.75)	0.42	0.41	0.31	0.32	0.28	732.5	14 200	726.8	726.8	5.40	732.1	731.2	731.3	730.9	
D-29	0.63	11.2	0.03	0.01	5	0.65	0.09	0.56	1.95	3.75	0.64	0.62	0.51	0.49	0.30	738.0	27 000	730.9	730.6	6.72	737.9	736.6	736.4	734.1	No scour.
31	0.87	20.8	0.46	0.44	52	0.93	0.56	0.37	3.62	.....	0.97	0.91	0.74	0.74	0.32	740.9	50 000	736.0	735.8	4.45	741.4	739.4	739.4	734.8	Few fines washed into pocket.
32	0.86	20.5	0.54	0.43	63	0.92	0.63	0.29	3.56	.....	0.96	0.90	0.74	0.73	....	740.8	49 400	737.0	735.7	3.88	741.8	739.4	739.3	.....	
33	1.27	34.8	0.93	0.95	75	1.39	1.08	0.31	6.06	.....	1.37	1.25	1.06	1.05	0.55	745.7	84 000	741.6	742.1	3.72	745.5	743.2	743.1	737.0	Some scour.
34	1.33	34.8	0.90	0.95	82	1.44	1.22	0.22	6.06	.....	1.37	1.25	1.10	1.05	....	746.5	84 000	742.8	742.1	2.64	745.5	743.7	743.1	.....	
35	0.37	4.50	—0.49	—0.43	0	0.38	—0.45	0.83	0.78	.....	0.35	0.34	0.27	0.27	0.25	734.9	10 800	724.6	725.4	10.0	734.6	733.7	733.7	733.5	No scour.
E-37	0.36	4.50	—0.53	—0.43	0	0.37	—0.48	0.85	0.78	.....	0.35	0.34	0.25	0.27	....	734.8	10 800	724.1	725.4	10.2	734.6	733.5	733.7	.....	
38	0.65	12.3	0.06	0.06	9	0.68	0.13	0.55	2.13	3.70	0.68	0.65	0.52	0.52	0.33	738.3	29 600	731.2	731.2	6.61	738.3	736.7	736.7	734.5	No scour.
40	0.87	20.0	0.40	0.41	46	0.93	0.50	0.43	3.48	3.86	0.94	0.89	0.71	0.72	....	740.9	48 200	735.3	735.3	5.16	741.2	739.0	739.1	.....	
41	1.05	25.8	0.70	0.64	67	1.13	0.81	0.32	4.50	.....	1.12	1.04	0.87	0.86	0.47	743.1	63 000	738.9	738.1	3.84	743.0	742.9	740.8	736.1	Some scour.
43	1.18	30.9	0.86	0.83	73	1.28	0.99	0.29	5.38	.....	1.26	1.16	1.03	0.96	0.52	744.7	74 200	740.8	740.5	3.43	744.4	742.7	742.0	736.7	Considerable scour.
48	1.28	34.0	0.98	0.94	77	1.39	1.11	0.28	5.91	.....	1.34	1.25	1.11	1.03	0.60	745.9	81 800	742.8	741.7	3.36	745.5	743.8	742.9	737.7	
F-44	0.65	11.9	0.01	0.04	0	0.68	0.08	0.60	2.07	3.72	0.66	0.63	0.52	0.51	0.32	738.3	29 600	730.7	731.1	7.20	738.1	736.7	736.6	734.3	Considerable scour.
45	0.87	4.70	—0.44	—0.42	0	0.37	—0.41	0.78	0.82	.....	0.36	0.36	0.31	0.28	0.25	734.9	11 300	725.2	725.6	9.38	734.8	734.1	733.8	733.5	Scour due to double jump.
48	0.65	12.3	0.05	0.05	8	0.68	0.12	0.56	2.14	3.82	0.68	0.65	....	0.52	0.33	738.3	29 600	731.1	731.2	6.72	738.3	.....	736.7	734.5	No scour.
49	0.89	20.2	0.43	0.42	48	0.95	0.53	0.43	3.50	3.81	0.95	0.89	0.70	0.73	0.35	741.2	48 800	735.7	735.5	5.16	741.2	738.9	739.3	734.7	Considerable scour.†
50	1.20	31.2	0.84	0.84	70	1.30	0.96	0.34	5.43	.....	1.27	1.17	1.03	0.97	0.52	744.7	75 000	740.6	740.5	4.09	744.5	742.9	742.1	736.7	
G-51	0.88	20.2	0.46	0.42	52	0.94	0.55	0.39	3.52	3.86	0.95	0.89	0.75	0.73	0.33	741.1	48 800	736.0	735.5	4.69	741.2	739.5	739.3	734.5	No scour.
52	1.01	25.3	0.62	0.62	61	1.09	0.74	0.35	4.40	3.85	1.10	1.02	0.86	0.85	0.41	742.6	61 000	738.0	737.9	4.21	742.7	740.8	740.7	735.4	No scour.
53	1.28	34.3	0.96	0.95	75	1.39	1.10	0.29	5.95	.....	1.35	1.23	1.12	1.04	0.59	745.9	82 600	742.0	741.8	3.48	745.8	743.7	743.0	737.6	Considerable scour.
55	0.73	14.5	0.20	0.17	27	0.76	0.27	0.49	2.52	3.80 (3.80)	0.76	0.73	0.58	0.58	0.32	739.3	35 000	732.9	732.5	5.89	739.2	737.5	737.5	734.3	No scour.

* Tests 15, 16 and 17 show effect of lowering tail-water. While adjusting flash-boards for Test No. 17, tail-water was lowered too far. This obliterated the jump and produced a smooth standing wave with trough over the gravel bed, which scoured the bed. No further scour resulted after tail-water was properly adjusted and jump re-established. † Scour due to double jump.

The various gauges used are shown in Fig. 58, and are described in Table 13. In addition to the gauges in Table 13 a profile board consisting of 2 by 6-in. timber was set on edge lengthwise over the center of the flume.

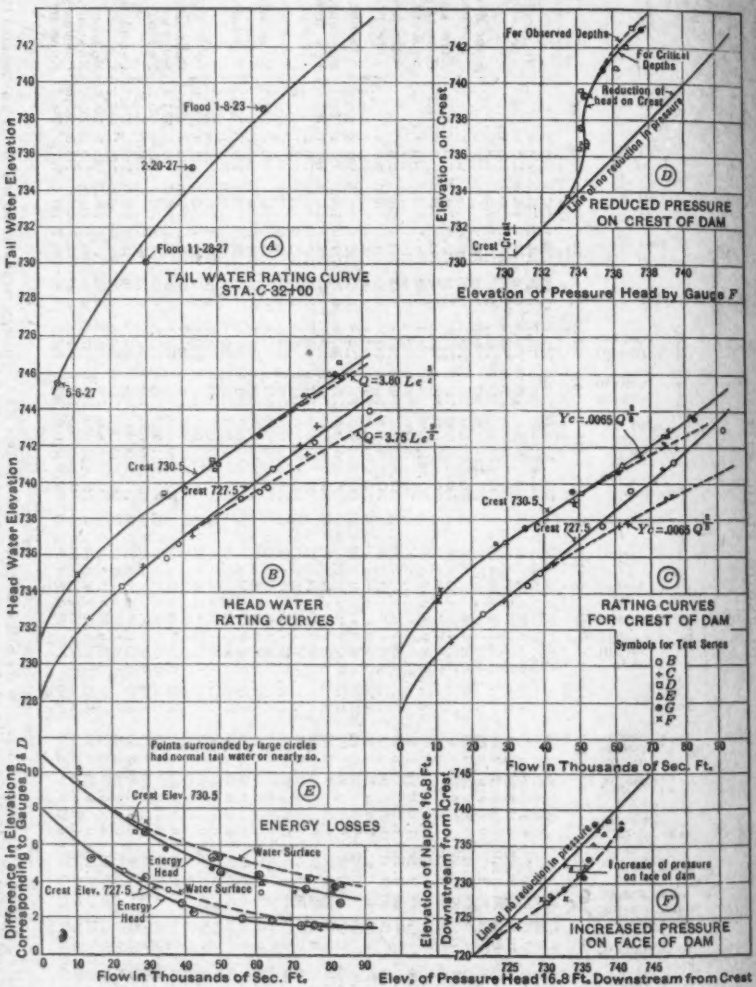


FIG. 62.—HYDRAULIC CURVES, LEABURG DIVERSION.

Distances were marked thereon up stream and down stream from the crest of the dam. Elevations were taken on each foot mark, and the longitudinal profile of the water surface was determined by measuring, with a level rod, the distances to the water surface from the top edge of the profile board.

Of the experiments made, five series were with open baffles; that is, square-faced piers with spaces between them, and one series with a solid baffle or weir the full width of the apron.

An examination of the profiles (Fig. 61) will show that in all cases where the open baffles were used the hydraulic jump occurs well on the apron. For low flows it occurs just over the baffles. As the flow was increased, it tended to move down stream, but remained on the apron.

The solid baffle in Series *F* invariably produced a double jump; the second occurred over the gravel bed, and was attended with more or less scour. The advantage of open baffles in controlling the position of the jump and in preventing scour is apparent, as it permits the use of a much shorter apron.

The arrangement of laminations used to produce various widths of baffles and openings had no effect that could be detected. This is to be expected since the retarding force is practically the same as long as the aggregate width of baffle and opening remains the same. For convenience of construction, baffles 4 ft. in width, with openings of like width between them, were adopted.

The upward slope of the openings between the baffles was 1 on 4 in all except Series *G*, in which a slope of 1 on 2 was used. No difference could be detected; hence, a 1 on 4 slope was adopted for ease of construction.

TABLE 13.—List of Gauges.

Gauge.	Kind.	Location and remarks.
D	Hook	In stilling well, with inlet 4.0 ft. up stream from crest of dam on left side of flume. Used to determine height of head-water.
B	Staff	In stilling well, with inlet 8.2 ft. down stream from crest, on left side of flume. Used to determine height of tail-water.
C	Staff	In stilling well, with inlet 3.7 ft. down stream from crest on left side. Used for tail-water height on apron. Installed after Run No. 17.
E	Staff	Piezometer connection on right side, 4.0 ft. up stream from crest. Used as check on Gauge *D*. Installed after Run No. 24.
F	Staff	Piezometer connection on right side of flume at center line of crest of dam. Used to determine pressure head on crest. Installed after Run No. 24.
G	Staff	Piezometer connection 1.4 ft. down stream on right side of flume and just above face of dam. Used to determine pressures on face of dam. Installed after Run No. 24.
H	Staff	Piezometer connection 4.5 ft. down stream from crest, on right side of flume just over gravel bed. Used to indicate pressures just below apron.
W	Hook	In stilling well in pond, 9 ft. up stream from rectangular weir. Used to determine head on the weir.

Fig. 63 shows a cross-section of the dam and a plan of the baffles as finally adopted for construction. It will be observed from Series *B* and *C* Fig. 61, that submergence of the dam begins with 15 000 sec-ft., but that the effect of submergence is not felt above the dam until a flow of 40 000 sec-ft. is reached. This is shown in Fig. 62 (*B*) for a crest elevation of 727.5.

The fall in the water surface and also the energy head lost as water passes the dam are shown in Fig. 62 (*E*). For the maximum flow of 90 000 sec-ft. the head lost is only 1.5 ft.; for the more frequent flood of 40 000 sec-ft., the drop in water surface is 3.1 ft.; and the energy head lost is 2.6 ft.

Some interesting data were secured on the cavitation effect as water passed the crest of the dam. Gauge *F* was a piezometer gauge in the side of the flume in line with the center line of the crest of the dam as shown in Series *C* and *D*, Fig. 61. It therefore registered the pressure head of the water at this point. The profile readings gave the actual depth at the crest. The difference is the reduced pressure or cavitation effect produced by the change of direction of water as it passed the crest. These data are shown at (*D*) Fig. 61. The difference amounted to 6 ft. for the higher flows.

The increased pressure on the down-stream face of the dam was similarly obtained by the profiles and piezometer, Gauge *G*. This increase in pressure is shown at (*F*) Fig. 62.

During tests with flows corresponding to 40 000 sec-ft., or more, a lead pencil, if placed beneath the nappe on the face of the dam, would play up and down on the face. In some instances its range of travel would be almost over the entire face from near the crest almost to the baffles. There was undoubtedly an oscillation of the vacuum near the crest and the increase of pressure on the face, that accounted for this phenomenon. It could not be due to eddy motion between the baffles and the face, for in that case the pencil would have risen into the nappe. Instead, it remained on the face sliding and rolling along it.

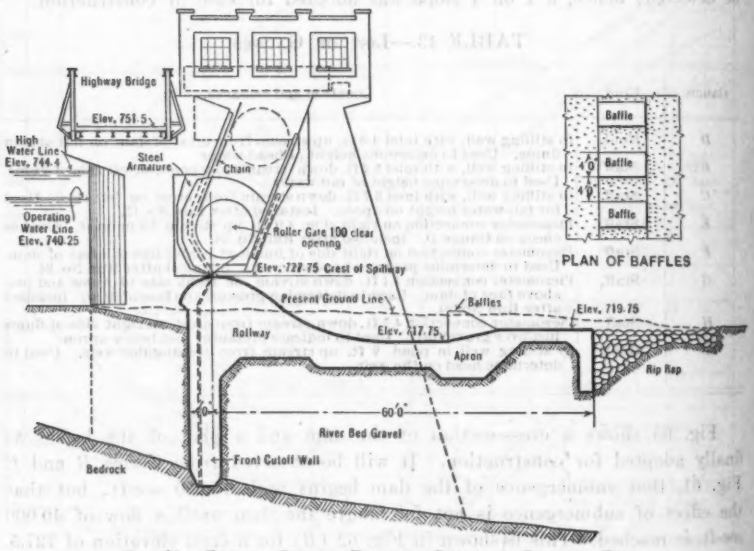


FIG. 63.—TYPICAL SECTION THROUGH SPILLWAY, LEABURG DAM.

During extreme floods the river gravels will be rolled along the bed whether or not there is a dam. If scour can be prevented at the immediate toe of the dam, it is the best that can be hoped. The experiments showed that with either type of baffles used, scour accompanied the formation of a

second wave beyond the apron, and that no scour occurred immediately below the apron. With a large flow and the tail-water held considerably lower than will actually occur, the water passed over the baffles as clear and smooth as if they were not there. A standing wave formed a considerable distance below the apron, that scooped a broad depression in the gravel bed.

In practice, such a condition can only occur if one large gate were wide open during a flood. Considerable experimenting was done with various gate openings and as a result the plan will be adopted of opening the three roller-gates together, to maintain the desired head above the dam during floods. If this plan had not been adopted, it would have been necessary to increase the height of the baffles more nearly in conformity with those used on the Pit River dams.

The writer believes the type of open baffle used on the Leaburg Dam will be more effective in maintaining a hydraulic jump on the apron than the truncated or sharp-nosed deflectors and piers used for the Pit River dams, and that either type is far superior to a solid weir.

I. C. STEELE* AND R. A. MONROE,† MEMBERS, AM. SOC. C. E. (by letter).—Floods of unusual magnitude occurred in Pit River during the latter part of March, 1928, and afforded the opportunity of observing and photographing the action of the baffle-piers at both Pit No. 3 and Pit No. 4 Dams. The maximum flow over Pit No. 3 Dam was approximately 15 000 sec-ft., with a depth of over-pour of 6.3 ft. (Fig. 64). At Pit No. 4 Dam the discharge was approximately 17 000 sec-ft. and the maximum depth of over-pour was 11.5 ft. (Fig. 65). The stilling action of the baffle-piers at both dams was highly satisfactory.

It should be observed (Fig. 66) that at Pit No. 3 Dam the two end splitter-piers are only half-piers and the sloping wing-walls were not provided with baffles. At the left wing-wall the water which was deflected from the end baffle-pier combined with the water passing directly between the half-pier and the wall and intermittently climbed the wall (Fig. 64). This action was expected, but is not considered to be of serious moment because the volume of water which rides the wall is quite small. It may be necessary at some future time to increase the side-wall protection at this particular locality by placing rip-rap on the bank above the existing concrete wing-wall.

At Pit No. 4 Dam it was observed that, for the flows noted, no better results occur on the depressed apron side, which indicates that the deeper stilling-pool is of little consequence. Unfortunately, the baffle-pier on the shallow side next to the right wing-wall was not constructed because of its location under the diversion flume. In addition, the right wing-wall is sloping as contrasted with the vertical side of the heavy division wall at the left side of the depressed apron. As a consequence there is created, along the right wing-wall, wave action which is not noticeable across the remaining width of the apron.

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An inspection was made of the baffle-piers and the apron below Pit No. 3 Dam shortly after the flood had subsided to determine what damage had occurred. Fig. 66 indicates the location and extent of certain pitting and erosion which had taken place since the completion of the structure. About midway between each of the splitter-piers small holes from 3 to 12 in. deep had developed apparently by the grinding action of rocks caught and held by the eddy which probably existed at this point. Approximately 75 cu. yd. of rock and débris were left on the apron at the completion of the construction work.

The faces and edges of the concrete piers show no erosion whatever except that two of the corners of the down-stream pier near the left wing-wall were chipped to a depth of 3 or 4 in., presumably by large rocks from slides which developed on the hillside above the wall. Several large rocks which evidently came from this location were found lying on the apron. These slides were due to a saturated condition of the hillside occasioned by heavy rains and, in time, this slope will become stabilized.

The efficiency of the baffle-pier designs in preventing erosion is indicated by the condition of the river bed immediately below the concrete apron. An area of limited extent on the right side has been scoured out approximately 1 ft. in depth. In the center of the channel, rocks and gravel have piled up several feet above the apron level. This material undoubtedly came from the débris which was left on the apron when the dam was constructed and from the slides on the hillside above the wing-wall, as previously noted. The area on the left side of the channel has not been disturbed materially.

The experiments mentioned by Mr. Ewald confirm the results of similar experiments in the Pit model tests. It is believed that baffles of the shapes and in the locations shown in Mr. Ewald's experiments might form an obstruction which would be subject to considerable shock. In the Pit River experiments it was soon observed that the best results were obtained by forcing the water to dissipate its energy within itself, the splitter and baffle-piers being placed rather far apart and acting more as guides to create cross-currents which would be directed largely against each other. Observations, although confined to the surface, indicated that the water throughout its entire depth was under great turmoil, and that the major portion of the kinetic energy was being destroyed by this action. Generally speaking, it is admitted that obstructions to the free and direct flow of the water away from the dam should be avoided, but it is obvious that special conditions encountered at a particular dam site must govern. As previously stated, the foundation and river-bed conditions of both Pit No. 3 and No. 4 Dams were quite unusual.

In the Pit model experiments the control weir mentioned by Mr. Ewald and others was placed at the lower end of the apron and at the point of free fall to the stream bed below. It was thought that this would simulate the natural barrier offered to flood waters by the stream bed below the aprons of the main structures. No such control weirs were actually constructed and, consequently, no secondary drop exists.

Mr. Scheidenhelm correctly states that "the most reliable criterion for comparison of the various means of preventing scour is the action on the

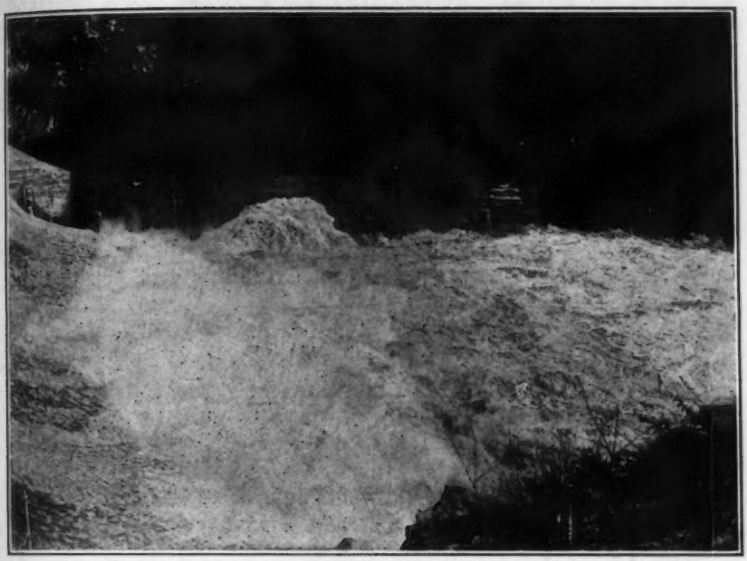

FIG. 64.—BAFFLE-PIERS AND APRON OF PIT RIVER DAM NO. 3, UNDER DISCHARGE OF 12 000 SECOND-FEET.

FIG. 65.—LOOKING DOWN ON BAFFLE-PIERS OF PIT RIVER DAM NO. 4, UNDER DISCHARGE OF 10 000 SECOND-FEET.

stream bed itself", and more conclusive evidence regarding the advantages or disadvantages of this design will be available after several years of actual performance. Results of three years' operations of Pit No. 3 Dam and one year of Pit No. 4 Dam indicate that the structures as designed are functioning in a very satisfactory manner.

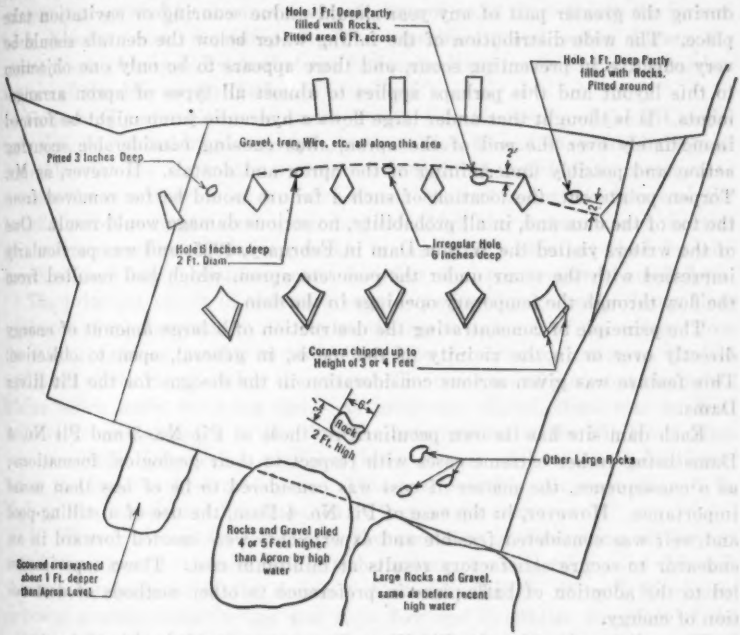

FIG. 66.—CONDITION OF THE APRON OF PIT NO. 3 DAM, MAY, 1928.

It is felt that the appearance of the surface and movement of currents of the water, a study of the action in and about the piers, and, what is perhaps more to the point, the smoothness of the flow immediately down stream, should afford a reliable indication of the effectiveness of the stilling device. A close study of the photographs of both tests and actual floods, such as Figs. 64 and 65, cannot but impress one with the excellent results from the standpoint of destruction of kinetic energy.

The experiments described by Mr. Sherman are very instructive. The principal differences between the design of the baffles of the Gatun Spillway Dam and those of the Pit River Dams, lie in the shapes and positions of the piers and in the bucket of the dam. It is thought that the Pit River design will cause a destruction of a greater percentage of the energy within the water itself. Published data on the Gatun Spillway were studied with a great deal of interest in connection with the preparation of designs for the Pit River Dams.

The data submitted by Mr. Torpen describing the tests for the Bull Run Dam are exceedingly interesting and enlightening and the plan adopted appears to afford a very satisfactory solution of a difficult problem. It would be very instructive to learn of the scouring effect of the fast moving water over the apron, particularly in the vicinity of the lower end of the bucket. No doubt, as in the case of the Pit River Dams, suitable repairs can be made during the greater part of any year should undue scouring or cavitation take place. The wide distribution of the falling water below the dentals should be very effective in preventing scour, and there appears to be only one objection to this layout and this perhaps applies to almost all types of apron arrangements. It is thought that under large flows a hydraulic jump might be formed immediately over the end of the apron, thus causing considerable scouring action and possibly undermining of the apron and dentals. However, as Mr. Torpen points out, the location of such a failure would be far removed from the toe of the dam and, in all probability, no serious damage would result. One of the writers visited the Wilson Dam in February, 1925, and was particularly impressed with the scour under the concrete apron, which had resulted from the flow through the temporary openings in the dam.

The principle of concentrating the destruction of a large amount of energy directly over or in the vicinity of the toe is, in general, open to objection. This feature was given serious consideration in the designs for the Pit River Dams.

Each dam site has its own peculiarities, those at Pit No. 3 and Pit No. 4 Dams being rather extreme cases with respect to their geological formations; as a consequence, the matter of cost was considered to be of less than usual importance. However, in the case of Pit No. 4 Dam, the use of a stilling-pool and weir was considered feasible and experiments were carried forward in an endeavor to secure satisfactory results at minimum cost. These experiments led to the adoption of baffle-piers in preference to other methods of destruction of energy.

The splitter-piers for the Pit River Dams are provided with 12 by 12-in. by ½-in. steel-plate noses, and no concern has been felt regarding the possible danger from floating logs. This might be more of a problem in other sections of the country.

The data submitted by Mr. Parshall relative to the Halligan Dam on the North Fork of the Cache La Poudre River are very interesting and indicate that in some cases the scour on an unprotected foundation below a relatively high dam may be of little consequence. The determining factor is without doubt the hardness or the resistance of the foundation to erosion. This cannot always be discovered by inspection, and, in some instances, over-pour dams founded on apparently hard rock have suffered extensive erosion. This particularly applies in cases where the over-pour is not confined to the center of the dam, but is allowed to spill across the entire crest length, in which case erosion of the canyon walls and the abutments is probable.

The description by Mr. Winter of the Martin Dam Spillway which embodies a return-flow conduit at the apron, is of interest. Doubtless such an installation will reduce the effective head at the end of the apron. The

directing of the water sheet downward at jump-off seems important to prevent the over-pour sheet from hurdling the piers and it is believed that for all flows over the Pit River Dams the water sheet will plunge into the basin waters rather than be deflected and perhaps carried along the surface. It is also necessary that there be sufficient relief below to permit the reversal of the flow mentioned by Mr. Winter. During the model tests the existence of a reverse current was verified by the movement of chips of wood thrown into the stream some distance below the jump-off. The experiments and action during the recent floods clearly indicate that the hydraulic jump more nearly approaches the toe of the dam as the discharge increases.

Undoubtedly a more precise means of recording the results of the Pit model tests than is afforded by photographs, such as Figs. 64 and 65, would have been desirable. At the time, this was not considered of sufficient importance to warrant the construction of a model stream channel below the dam and the installation of accurate measuring devices. Also, the storage above the test dam was too small to make it possible to maintain a constant head for any length of time over the spillway crest for other than comparatively small flows.

The principal object of the tests was to confine the hydraulic jump close to the toe of the dam, and to destroy on the apron as much of the energy from the falling water as possible in order to reduce the scouring action on the bed and banks of the stream below the apron. The two rows of piers tend to create basins which assist in doing this. As previously stated, there was no final back-water weir although at the time the construction of the Pit No. 3 Dam was completed there was a very low natural weir of boulders immediately below the lower end of the apron. This was probably not more than 3 ft. high on the average and has since partly been swept away.

Mr. Winter's remarks relative to the necessity of dissipating all energy within the water if the structure is to be lasting, are well taken, and this was the primary purpose of the design. A study of Figs. 64 and 65 clearly indicates a tremendous boiling and cross flow and illustrates the effect of the water impinging on itself and a comparatively even flow at the lower end of the aprons. All the energy cannot, of course, be so dissipated and some wear or scouring of the concrete is anticipated. In this regard it is interesting to note that during the model tests only two six-penny nails were required to hold the curved baffle-piers in position on the apron. However, in the final plans, the concrete piers and apron were designed and reinforced to withstand the full force of the water sheet across the entire cross-section area of the piers. This did not lead to any unusual or extreme construction. A part of the apron slab was designed to act with the pier to prevent possible over-turning. No fear is felt that the apron will be broken up by the action of the water. Should any of the concrete protective work show signs of dangerous wear, it is accessible for repair during the major part of the year, as the power house takes the entire river flow, except during flood periods in the spring months.

Mr. Bauman has submitted a very interesting mathematical analysis of a subject which is particularly difficult to treat analytically. His basic formulas for determining the length of river bed which should be protected

below a dam are well established and should be of assistance to engineers who in the future may be confronted with problems of this character. In particular, his method will be of value in pointing the way for further research, but in the final analysis it is believed that investigation by means of models and observation of completed structures will be necessary to obtain data which can be relied on with certainty.

Reference to the notched or dental weir method of protecting the down-stream channel from scour has been made by several discussors and there seems no doubt that this device might be used to advantage in many cases.

Mr. Stevens has submitted an enlightening discussion particularly in that he presents data obtained from a number of tests on models using the dental or notched weir to prevent erosion. Apparently, the situation at Leaburg had been carefully analyzed before deciding to use the dental weir to the exclusion of other methods of preventing scour. This series of experiments is especially noteworthy in that data were secured giving the complete profile of the water surface under the various tests. The effect of the scour on the stream bed was also determined. It is to be noted that the solid weir did not give as satisfactory results as the notched type, which fact could be inferred from the Pit River model tests. The data obtained as to the height and location of the dentals to give the best results should be of assistance to future investigators. The observation that a considerable percentage of submergence was necessary before the up-stream water level was affected, was to have been expected from results obtained in other investigations, but might easily be overlooked by an engineer not familiar with this phenomenon.

In conclusion, it might be observed that each discussor presents some particular method of handling the problem of scour below dams, but that none of the discussors apparently has utilized a baffle-pier design similar to the type used below the Pit River Dams. The basic idea of the Society's Special Committee on Irrigation Hydraulics in having this paper published was to bring out additional information regarding other successful methods of preventing scour below dams, and it is felt that in the many discussions offered much valuable data have been accumulated and that a great deal of attention is being given to this problem.

The authors wish to express their appreciation for the many contributions to their paper. Acknowledgments are also due the following engineers who were in immediate charge of the tests: Walter Dreyer, Assistant Engineer in Charge of Design, Grover C. Green, Field Engineer, Clinton de Witt, Assistant Engineer, and Arthur W. Kidder, Assistant Field Engineer, all Associate Members of the Society. The engineering for Pit No. 3 and Pit No. 4 projects was under the direction of A. H. Markwart, M. Am. Soc. C. E., Vice-President in Charge of Engineering, Pacific Gas and Electric Company. John D. Galloway, M. Am. Soc. C. E., was Consulting Engineer on both dams.

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Paper No. 1707

METHODS USED IN THE CONSTRUCTION OF TWELVE PRE-CAST CONCRETE SEGMENTS FOR THE ALAMEDA COUNTY, CALIFORNIA, ESTUARY SUBWAY*

By Alvin A. Horwege,† Assoc. M. Am. Soc. C. E.‡

With Discussion by Joseph A. Kitts, Assoc. M. Am. Soc. C. E.

Synopsis

This paper describes in detail the construction of the twelve pre-cast concrete segments of the Estuary Subway between Oakland and Alameda, Calif. It includes a brief statement of the nature of the work and of general conditions, a description of the mixing plant, distributing system, and forms, and the method used in pouring the different sections of a segment.

Purpose of Construction

The site of the Oakland-Alameda Estuary Subway was formerly occupied by the railroad bridge of the Southern Pacific Railroad Company and used as a suburban connection between Oakland and the Alameda Pier, and is within several hundred yards of the highway and street-car bridge connecting Oakland and Alameda.

Because of the heavy increase of the shipping in the Oakland Estuary these bridges had become a serious menace to the traffic in the Upper Estuary and, at the solicitation of the War Department Engineers, Alameda County began an investigation which resulted in the approval of plans and the voting of bonds for the construction of an estuary subway which will eliminate both bridges and give the Upper Estuary Harbor an unobstructed channel for shipping.

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† Supt., California Constr. Co., San Francisco, Calif.
‡ Mr. Horwege resigned from the Society on February 9, 1929.

Nature of Work

Early in 1925, the California Bridge and Tunnel Company was awarded a contract to construct the Estuary Subway between Oakland and Alameda, Calif. In general, the work consisted of two open-cut approaches totaling 930 ft.; two portal buildings totaling 136 ft.; a rectangular section on the Oakland side, 342 ft. long; a transition section, 37 ft. long, connecting the rectangular to the circular section; a circular section built in place on the Oakland side, 578 ft. long; and twelve pre-cast concrete tubes, each 203 ft. long, to be built in Hunters Point Dry Dock, floated across the Bay, sunk in place, and joined under water.

General Conditions

Time was an important element in the construction of these tubes. The Bethlehem Shipbuilding Corporation was reluctant to permit the dock to be tied up a reasonable length of time, from the standpoint of economy to the contractors. The rental of the dock was limited to six months, after which a severe penalty was to be added to the contract rental. The result was that the contractors were forced to provide a rather elaborate plant, considering the quantity of concrete to be poured—29 000 cu. yd.—and to use steel forms throughout. To permit rapid erection, three full shifts of men were employed during the construction period.

Delivery of Material

Practically all the material, except the cement, was delivered in barges. (See Fig. 1.) The cement was hauled from the railroad, a distance of 2 miles, in trucks as it was used. At first an attempt was made to store a reserve supply on the job, but because of rehandling, this proved more costly than to haul as it was needed.

The rock and sand were delivered in barges of 400-ton capacity and unloaded with an electric hoist and steel derrick equipped with a 60-ft. boom and a 1-yd. clam-shell bucket. The material was dumped into a receiving hopper at the end of the dock, from which it was conveyed by a system of 24-in. belt conveyors to a storage pile. These piles were confined by wooden retaining walls filled with the material to give them stability. The first conveyor was 94 ft. long, and dumped on the second conveyor, 114 ft. long, at an angle of about 15°; and this, in turn, delivered to a movable distributing conveyor, 40 ft. long, which spread the material in piles, about 75 ft. long, at right angles to the belt conveyor. The sand was segregated from the rock and, of course, could not be conveyed from the barge at the same time as the rock.

Mixing Plant

Adjacent and parallel to the stock piles a ramp, 20 ft. high, was erected, from which a crane, fitted with a clam-shell bucket, fed the hoppers on top of a two-story mixing house. (See Fig. 1.) Two 1-yd. concrete mixers were placed on the ground floor of the mixing house. Batchers for the rock, and inundators for the sand, a separate set for each mixer, were placed on the second floor. The inundators, because of the almost uniform moisture content

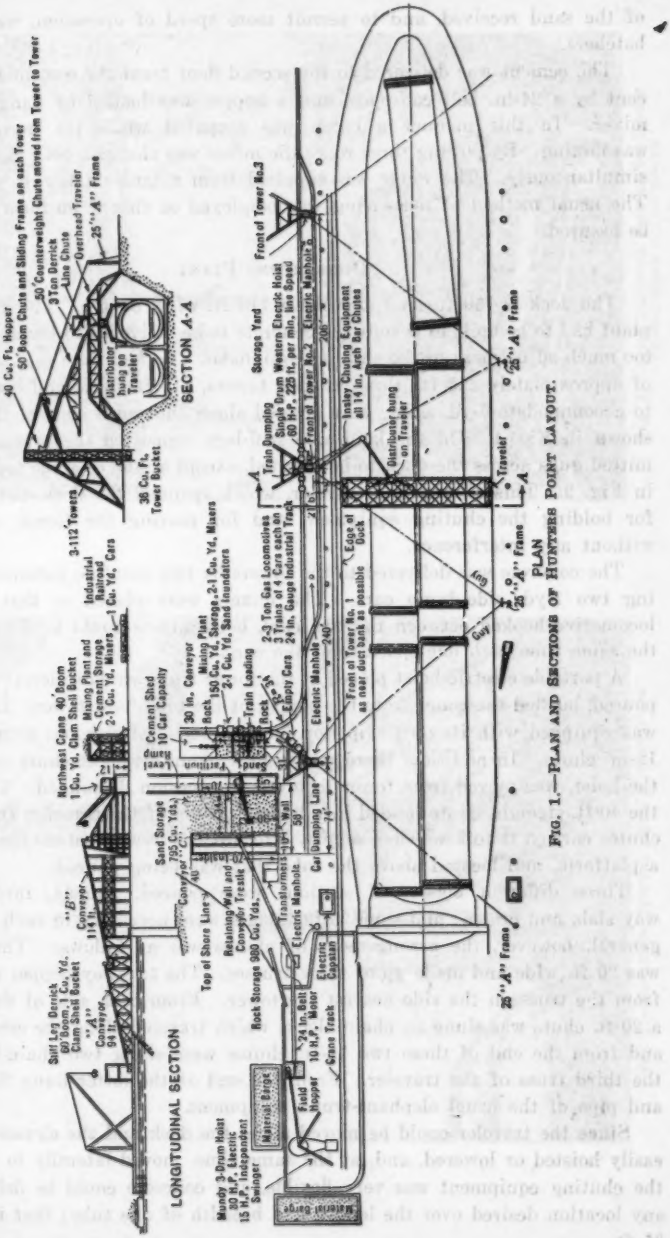

FIG. 1.—PLAN AND SECTIONS OF HUNTERS POINT LAYOUT.

of the sand received, and to permit more speed of operation, were used as batchers.

The cement was delivered to the second floor from the cement house adjacent by a 24-in. belt conveyor, and a hopper was loaded by hand above the mixer. In this manner a batch was prepared while the previous batch was mixing. By pulling three ropes the mixer was charged, both mixers acting simultaneously. The water was supplied from a tank on top of each mixer. The usual method of measuring was employed so that a uniform mix could be assured.

Distributing Plant

The dock is 750 ft. long and about 110 ft. wide, on top. The distributing plant had to be built to permit the concrete to be delivered to the dock without too much adjustment of the chuting equipment. To facilitate this, at intervals of approximately 225 ft., three wooden towers, 112 ft. high, and large enough to accommodate 1-yd. skips, were erected along the south edge of the dock as shown in Fig. 1. On the land side stiff-legs supported the towers and permitted guys across the dock to be omitted, except at the extreme top as shown in Fig. 2. This allowed the traveler, which spanned the dock and was used for holding the chuting equipment and for moving the forms, to operate without any interference.

The concrete was delivered to the towers by two gasoline locomotives hauling two 1-yd. side-dump cars. The mixers were placed so that with the locomotive hooked between the two cars, both mixers could be discharged at the same time with one spotting of the cars.

A portable electric hoist placed at the tower from which concrete was being poured, hoisted the concrete to the hopper at the top of the tower. Each tower was equipped with its own skip, top hopper, and a steel boom with 40 ft. of 12-in. chute. In addition, there was a 40-ft. counterweight chute which, like the hoist, was moved from tower to tower as occasion demanded. The end of the 40-ft. straight chute landed in a hopper on top of the traveler, from which chutes carried it to a wooden hopper, fitted with two-way gates operated from a platform, and located above the tube that was being poured.

Three different classes of sections were poured, namely, invert, roadway slab, and crown; and some adjustments were necessary in each case. In general, however, the arrangement of chutes was as follows: The traveler was 20 ft. wide and made up of three trusses. The two-way hopper was slung from the truss on the side nearest the tower. From each side of the hopper, a 20-ft. chute was slung on chain-blocks which traveled along the center truss, and from the end of these two 15-ft. chutes were slung two chain-blocks on the third truss of the traveler. From the end of the latter hung the hopper and pipe of the usual elephant-trunk equipment.

Since the traveler could be moved along the dock and the chutes could be easily hoisted or lowered, and, at the same time, moved laterally to the dock, the chuting equipment was very flexible, and concrete could be delivered to any location desired over the length and breadth of one tube; that is, 203 by 35 ft.

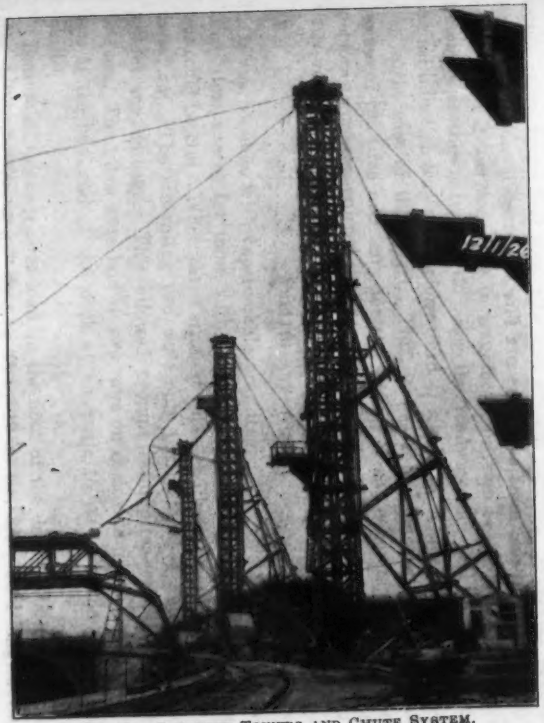

FIG. 2.—CONCRETE TOWERS AND CHUTE SYSTEM.

FIG. 3.—VIEW SHOWING SECTION OF INSIDE CROWN FORM BEING MOVED.

Concrete

Table 1 gives the various gradings for the Class "A" mix used in the construction of the tubes, proportioned for 1 cu. yd. of concrete in place and certain characteristics based on a series of tests.

TABLE 1.—Concrete Data.

Grading, *n*	0.53	0.54	0.55	0.56	0.57
Fineness modulus of sand	5.87	5.91	5.95	5.99	6.03
Cement, sacks	8	8	8	8	8
Sand, in cubic feet	11.19	11.7	11.4	11.2	11.9
Rock, in cubic feet	21.0	21.4	21.8	22.2	22.6
Ratio of cement to voids	1.28	1.26	1.24	1.21	1.19
Ratio of sand to voids	1.14	1.09	1.05	1.01	0.98
Ratio of mortar to voids	1.96	1.91	1.86	1.81	1.76
Density	0.82	0.825	0.833	0.842	0.85

A shrinkage of 3% in setting was allowed, and the proportions produced not less than 1 cu. yd. of concrete in place.

The grading, *n* (0.55), and the fineness modulus (5.95), were most satisfactory for the conditions and, with a slump of from 6½ to 7½ in., the concrete developed a strength of 3 000 to 3 500 lb. in 28 days.

Forms

On the Hunters Point work the inside crown forms were of steel and the outside crown forms had steel ribs and wooden lagging. Wherever possible, forms were mounted on trucks and ran on tracks from one tube to the next, except where it was necessary to transfer them from one side of the dock to the other, as shown in Fig. 3. For the roadway slab and the crown, the original intention was to pour one-third of a tube at a time and the complete tube in the case of the invert. This, it was found, caused too much delay because of the time necessary for the setting of the concrete, and additional steel forms were constructed where wooden forms could not be used, and wooden forms were built on the job in the other instances.

Preliminary Work

It was possible, with considerable crowding, to lay five tubes in the dock at one time, except for those with horizontal curves, in which instance only four could be laid. When three straight tubes had been completed, they were floated out. The remaining two were only partly finished and were flooded without any damage. This allowed prompt resumption of the work after flotation. The tubes were laid in two rows on each side of the dock, with one tube at the bow end of the dock at an angle to its axis, the others being parallel.

Because of the enormous load on the floor of the dock, 5 000 tons for each tube, the weight was distributed by 12 by 12-in. timbers placed longitudinally on 2-ft. centers, on the floor of the dock and shimmed where a depression did not permit full bearing. Across this layer were placed 12 by 12-in. timbers, 38 ft. long, spaced to conform with the invert forms and, roughly, on about 3½-ft.

centers. These timbers were leveled carefully at a fixed elevation so that calculations for the setting of forms were the same for all tubes. Great care was taken to provide uniform bearing in this case and thus prevent excessive settlement. Fig. 4 shows the subsequent stages of construction work herein described.

Saddles

The center portion of the tube for a width of 10 ft., 1½ in. on each side of the center line, was supported on wooden saddles which remained in place until the tube was floated. The center section was made from 4 by 16-in. timbers, 7 ft. long. Bolted to these at one end and to 6 by 12-in. posts at the other, were secondary saddles made from 4 by 12-in. timbers. They were also supported twice in the middle of the span with 4 by 6-in. posts. All saddles were cut to conform with the outside curvature of the completed tube. The secondary saddles were unbolted just prior to floating the tubes and were pulled from under the tube afterward. This was necessary in order to provide clearance for the end bracket at the end of the tubes. Fig. 5 is a view of a lower cradle and outside form, showing the method of erection.

Outside Invert Forms

On each side of the wooden saddles were placed the outside invert forms. These were steel ribs built as cantilevers and tied together into 19½-ft. sections. There were seven ribs with wooden nailing strips fastened to curved angles at 3 ft. 3-in. centers. Between these were wooden ribs, 2 in. wide, to reduce the span. (See Fig. 5.) Each invert section was separated from the next by 22-in. spacing which permitted them to be swung, in the case of the horizontal curves, without interference.

After these invert forms were set in place with cranes, they were lined and leveled by hard-wood wedges on top of 8 by 12-in. longitudinal timbers and braced with struts to the floor of the dock. The lagging of 2 by 12-in. material was next laid on top of the invert forms and the wooden saddles up to the horizontal diameter. As this lagging served a double purpose, that of the outside forms and the protection for the water-proofing, and remained on the tube, it was not nailed to the form ribs except to draw a warped end down. Outside invert forms were provided for two complete tubes.

End Brackets

At each end of the tube the design specified an end bracket, which consisted of a square block of concrete through which the circular portion of the tube passed, as shown in Fig. 6. These served a double purpose; that is, as bases for landing the tubes in place on concrete blocks, and for joining the tubes. The forms for the end brackets were of steel, except for the wood bottom which remained in place until the tube floated off it. There were four complete forms for the invert; two inside rings; two outside sets of plates up to a little above the horizontal diameter; and one form for the top outside and the end face.

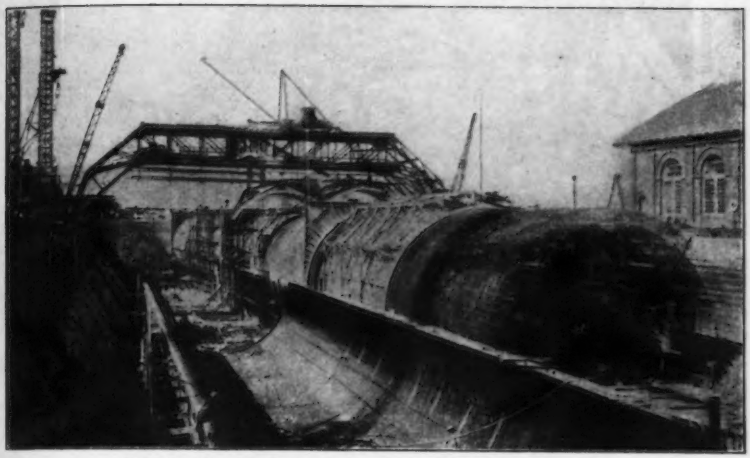

FIG. 4.—VIEW SHOWING VARIOUS STAGES OF CONSTRUCTION.

FIG. 5.—VIEW OF LOWER CRADLE AND OUTSIDE FORM.

FIG. 6.—PRE-CAST SEGMENT LEAVING HUNTERS POINT DOCK.

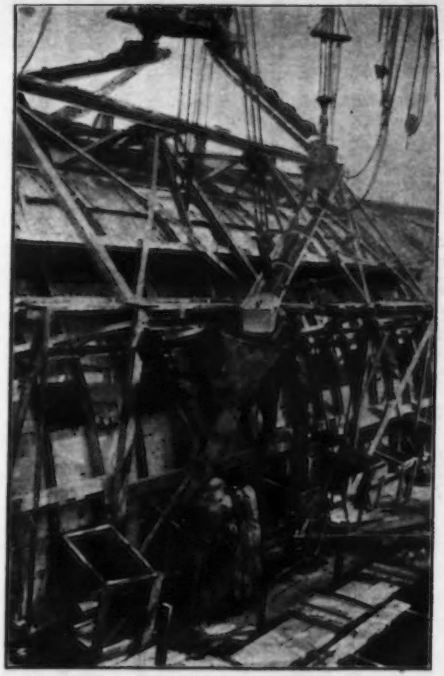

FIG. 7.—POURING CONCRETE IN CROWN SECTION.

Invert Water-Proofing and Steel

After the invert lagging and the end bracket invert forms were placed, the water-proofing for the tube, below the horizontal diameter, was laid on the lagging and ended by metal flashing in the end brackets. The water-proofing consisted of three layers of cotton fabric in hot asphalt. The bottom 20 ft. was next covered with a 1-in. layer of grout to protect the water-proofing from the reinforcing steel that was next placed. The steel was set on 2-in. concrete blocks 2½ in. high. It was found that the weight of the steel forced the blocks through the water-proofing unless it was protected.

Inside Invert Forms

The inside invert forms were made of steel and were hung to the outside forms by means of steel brackets and bolts. The bolts passed through the water-proofing, the holes being later patched. They extended from a point approximately level with the bottom of the roadway slab to a point 8 ft. from the center of the tube. This left an opening 16 ft. wide in the bottom of the tube, which was supposed to be screeded to conform to the inside curvature. Because it was necessary to pour the concrete rather wet so that it would flow properly around the network of steel, it was found that the steel forms had to be supplemented with wooden sections 4 ft. wide to reduce the opening to 8 ft. Both the steel and wooden forms were securely fastened with ⅞-in. bolts to the reinforcing steel and rested on the 2½-in. concrete blocks on top of the steel. Four of the tubes had an 8-in. longitudinal wall under the center of the roadway slab. A projection, 16 in. high, was formed in this opening. Otherwise, the concrete was screeded and finished smooth while being poured. The form sections were 22 ft. long and a sufficient number for one complete tube was provided.

Invert Concrete

The entire invert, which contained about 715 cu. yd., was poured continuously and required about 11 hours to complete. For the first half of the tube, the bottom was filled to the lower part of the form. The chutes were then changed to the outside and the sides filled to the top. At the same time the concrete would run forward and fill the bottom ahead of that previously poured. Sometimes it was found advisable to switch back and fill the bottom for the remainder of the distance, but the bulk of the concrete was poured at the top and allowed to run down along the outside forms. Before the top was set too hard it was worked by hand to conform to the longitudinal joint prescribed by the County Engineer. This included a key to which was fastened either a copper or galvanized water-stop 10 in. wide, one-half of which was in the invert concrete. The forms were stripped as soon as the concrete had set sufficiently and all irregularities were smoothed off and plastered.

Roadway Slab Forms

Within 10 hours after the invert was poured four rails were laid on the concrete on longitudinal stringers properly spaced and braced to take the trucks

of the roadway forms. These forms were then rolled out of the previous tube on to this track and jacked up to grade. The forms consisted of three sections, 21 ft. 8 in. long, mounted on eight trucks and made of steel. Between them were suspended two sections 16½ ft., made of wood, on the job, and sheathed with No. 10 gauge plates. Each section was separated along the center line and, in the case where the center wall was under the road slab, vertical forms with suitable turnbuckles for adjustments were suspended to these forms. In other cases, the opening between them was covered with a No. 10 gauge plate. On the outside edge were hinged wing forms which were swung up against the invert concrete by turnbuckles and then securely held by wooden struts. The wooden forms were held by wooden posts and wedges.

As soon as the forms were set to grade the road-slab steel was placed and side longitudinal forms of wood set in between the steel were built. Screeds, all the interior fixtures, and bulkheads at the end, were placed and the concrete for one-half the tube was poured. This amounted to about 158 cu. yd.

The forms were allowed to stand for 48 hours when they were collapsed and moved either to the other half of the same tube, or to the next tube.

Reinforcing Traveler for Crown Steel

Twenty-four hours after the road slab was poured, two 50-lb. rails were laid on top of longitudinal stringers. The reinforcing traveler was then moved on to the road slab and jacked into place. This traveler consisted of four sections, each 20 ft. long. Three sections were mounted on trucks, and the fourth was suspended by 12 by 12-in. stringers between two of those on the trucks. This arrangement was used only when moving; when spotted, the fourth section was jacked up directly from the road slab. The others had jacks mounted on the trucks. All were provided with turnbuckles for spreading to the proper width, and collapsing after the steel was in place.

Each section was made up of a steel bed-frame, on which were mounted three wooden lateral trusses. Running longitudinally were metal angle-bars at intervals of about 10 ft., which later were supplemented by wooden members between, and all built to conform to the prescribed radius of the reinforcing steel, that is, 16 ft. 2½ in. Each section was separated about 5 or 6 ft. according to the needs and closed with spliced angles. Fig. 4 is a view of the reinforcing steel traveler with crown steel in place.

When the traveler was set to grade and spread to the proper width, the arch or crown reinforcing steel was placed on it. As in the invert, there were two rings of this tied together with longitudinal steel and also to the invert rings which protruded from the invert concrete. The ring steel varied from ¾ in. to 1 in., according to the tube, and was spaced on 12-in. centers. The longitudinal steel was ¾ in. throughout and was spaced on 6-in. centers.

Outside Crown Forms

The original outside crown forms were made up of three sections each 21 ft. long. Each consisted of two pairs of trusses hinged at the top and heavily cross-braced between. To this frame was bolted 3 by 12-in. wooden lagging

When the length was increased to make possible the pouring of one-half the tube instead of one-third, two more 15-ft. sections were built of similar design. Four rows of doors were provided on each side through which the concrete could be poured and a 10-ft. space, where there was no lagging, was left at the top.

The bottom rested on top of the outside invert forms at the horizontal diameter and was bolted securely to it. Just above this point holes were provided to allow a row of 1¼-in. bolts to pass to the inside crown forms. These were spaced about 4-ft. centers and were the only lateral support for these forms. When these forms were set, hook-bolts were passed through holes provided at intervals and hooked to the reinforcing steel and tightened. The reinforcing traveler was then collapsed and moved out to the next set up. Fig. 4 shows a view of an outside crown form in place; also, a section of the outside crown form being lifted and moved with the gantry.

Inside Crown Forms

The inside crown forms were next moved into place under the reinforcing steel which was suspended from the outside crown forms by the hook-bolts. These forms consisted of three sections, 23 ft. long, each mounted on trucks, as shown in Fig. 3. Two sections, 15 ft. long, were built in San Francisco, Calif., when the "pour" was increased, and were suspended between the original sections. The first three sections were provided with jacks for raising and steamboat ratchets for spreading to the proper width. The others had no such mechanism, but when bolted to the first were rigid enough to follow the others. All were hinged at the top.

The general design consisted of a heavy bed-frame mounted on trucks with two heavy pairs of trusses rigidly cross-braced and covered with No. 10 gauge plates. A heavy waler ran longitudinally just above the horizontal diameter to take the 1¼-in. through bolts from the outside crown forms. Near the bottom was a hinged wing form. When moving, this was hooked up and kept there until the form was set at the proper grade, and spread when it was lowered to a sloping wooden filler strip. Struts were then placed from the track stringers to the base of these wing forms and wedged, and the jacks were slacked off. The forms then rested on the sloping filler pieces and became a rigid self-contained unit.

After the 1¼-in. through bolts were set, the interior fixtures placed, and a bulkhead was provided at the center of the tube, the forms were ready for concrete. Horizontal and vertical curves were formed by means of tapered filler pieces between the sections of forms. These were carefully calculated and the forms had to be spotted accurately to make them conform to the required curvature.

End Bracket Form

As soon as possible after the outside crown forms were placed, the back top of the end bracket form was set. For ease in handling, this was divided

into three pieces and the reinforcing steel for the portion was placed against it. The front section was next placed and the form was ready for pouring.

Crown Section

The crown section amounted to about 700 cu. yd., for one-half the length of the tube and was poured in about 12 hours. This was divided into three operations. The first was through the lower row of doors. (See Fig. 7.) The steel was spread to allow the hopper chutes to be inserted into the center of the wall. These hopper chutes had a bend in the spout so as to turn the concrete in a vertical direction and prevent the rock from hitting the steel and separating from the mortar. There were 13 to 14 chutes on each side, placed directly opposite in pairs. Ten men on a side were stationed between the forms to tamp and spread the concrete while it was being poured. When the forms were filled to the row of doors, the hoppers and chutes were raised to the next row, the lower doors were closed, and the process was continued. When the forms were filled to the second row they were closed and the chutes were raised to the top opening and the remainder of the concrete was poured from that point. The top of the end bracket was poured last. Six men with air hammers vibrated the inside crown forms during the pouring.

Forty-eight hours after completing the "pour" the outside crown forms were moved to the next position. The outside invert forms were moved at this time, also. Twenty-four hours later the inside crown forms could be moved.

Water-Proofing and Lagging

As soon as the outside crown forms were moved men with chipping hammers smoothed up the concrete so that the water-proofing would not be injured. Scaffolding was then set up and the upper half of the tube was water-proofed. The lagging, for protection, was then installed and hoop rods every 3 ft. held it in place. Temporary wooden forms were set up against the lagging below the horizontal diameter to hold that lagging against the concrete after the invert forms were removed and before the hoop rods could be placed all around the tube. As soon as the rods were tightened these wooden forms were removed.

Concrete Bulkheads

Bulkheads were built to close the ends of the tubes for purposes of flotation and submersion. Below the roadway slab a heavily reinforced concrete bulkhead was built. Wooden forms were used in this work.

Wood and Steel Bulkheads

Above the roadway slab the construction was varied to suit the pressure. Under high heads 24-in. steel I-beams were set vertically at 1-ft. centers, against a shoulder provided in the end bracket. They were covered with 6 by 8-in. timbers, laid flat, and then sheathed with 1 by 6-in. tongue-and-groove stock. The whole was water-proofed, the edges being run into a keyway pro-

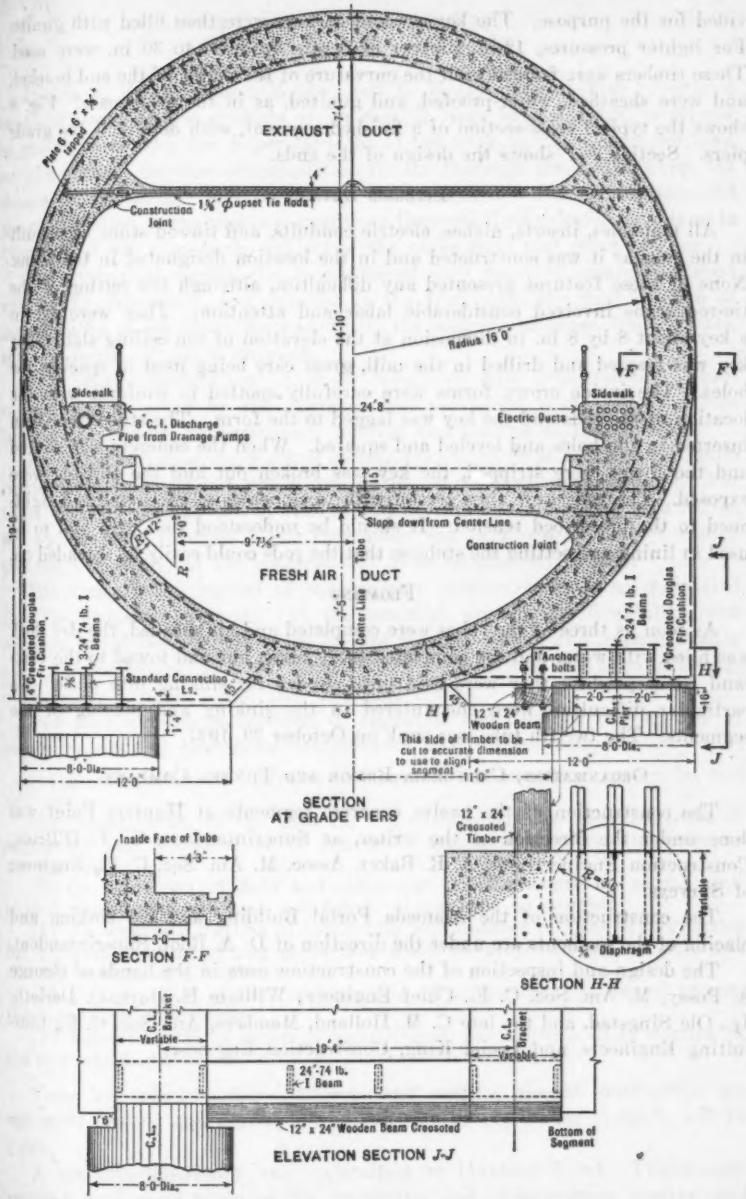

FIG. 8.—TYPICAL SECTIONS OF PRE-CAST SEGMENTS.

vided for the purpose. The keyway and corners were then filled with gunite. For lighter pressures, 12-in. timbers of depths from 20 to 30 in. were used. These timbers were framed to fit the curvature of the inside of the end bracket, and were sheathed, water-proofed, and gunited, as in the other case. Fig. 8 shows the typical cross-section of a finished segment, with details at the grade piers. Section *F-F* shows the design of the ends.

Interior Detail

All manholes, inserts, niches, electric conduits, and tie-rod stubs were built in the tube as it was constructed and in the location designated in the plans. None of these features presented any difficulties, although the setting of the tie-rod stubs involved considerable labor and attention. They were set in a key about 8 by 8 in. in dimension at the elevation of the ceiling slab. The key was framed and drilled in the mill, great care being used in spacing the holes. The inside crown forms were carefully spotted to conform with the location of the rods and the key was lagged to the form. The stubs were then inserted in the holes and leveled and squared. When the concrete was poured and the forms were stripped, the key was broken out and the threads were exposed. The rods were then set up with turnbuckles at each end and tightened to the prescribed tension. It should be understood that care had to be used in lining and setting the stubs so that the rods could easily be threaded on.

Floating

As soon as three of the tubes were completed and bulkheaded, the dry dock was filled with water and the completed tubes taken out and towed to the Oakland Estuary where they were held until needed for sinking into place. No particular difficulties were encountered in the sinking and placing of the segments. The twelfth tube was sunk on October 29, 1927.

Organization, California Bridge and Tunnel Company

The construction of the twelve concrete segments at Hunters Point was done under the direction of the writer, as Superintendent, G. T. O'Brien, Construction Engineer, and A. R. Baker, Assoc. M. Am. Soc. C. E., Engineer of Surveys.

The construction of the Alameda Portal Building and the sinking and placing of the segments are under the direction of D. A. Root, Superintendent.

The design and inspection of the construction were in the hands of George A. Posey, M. Am. Soc. C. E., Chief Engineer; William H. Burr, C. Derleth, Jr., Ole Singstad, and the late C. M. Holland, Members, Am. Soc. C. E., Consulting Engineers, and Locial King, Construction Engineer.

DISCUSSION

JOSEPH A. KITTS,* ASSOC. M. AM. SOC. C. E. (by letter).—This paper is a valuable record of the methods of pre-casting large tube sections, as viewed by the engineer who was Superintendent of the work for the Contractor, and he (the author) deserves commendation for the thorough manner in which the details of equipment and operations are presented. The writer observed the construction of the twelve segments as Concrete Technologist for Alameda County and will discuss the subject of the paper from that point of view.

The cylindrical-shaped, closely reinforced shell required concrete of good workability, flowability, and cohesion in the fresh mix; the final location of the structure in sea water demanded that the concrete be dense, impermeable, and, of course, of ample strength.

With due consideration of these conditions and requirements, ample cement was specified by the engineer and provisions were made for controlling the testing, proportioning, mixing, and placing of the concrete.

For Class "A" concrete, 2 bbl. of cement per cu. yd. in place and a minimum compressive strength of 2 500 lb. per sq. in., at 28 days, were required. For Class "B" concrete, 1½ bbl. of cement and a compressive strength of 2 000 lb. per sq. in. were required. The contractor was held responsible for results and the workability of the concrete had to be satisfactory to him.

The aims in the control of the concrete production were, to maintain: (*a*) The required strength; (*b*) the essential workability, flowability, and cohesion in the fresh mix; (*c*) the highest possible density and impermeability under the conditions; and, (*d*) a uniform concrete with a minimum of honeycomb seams between pours, and shrinkage cracks.

New developments in methods of aggregate and concrete testing, and of proportioning, and production control were used. These comprise:

(1) A comprehensive physico-mathematics of concrete materials and mixtures.
(2) Co-ordination of the tests for the various physical characteristics of aggregates, and co-ordination of aggregate, aggregate mixture, and concrete mixture tests.
(3) Co-ordination of field and laboratory measurements by actual, dry-rodded, loose-measured, and inundated volume, and by weight.
(4) An algebraic method of combining any number of sizes of aggregates for uniform grading of any coarseness modulus, proportioning by size, and absolute volume of particles.
(5) Co-ordination of the fundamentals of the various theories of proportioning concrete mixtures.
(6) Establishment of constant technological control as a daily routine of the concrete manufacture.

These are comprehensive subjects and only a general outline of the aggregate testing and aggregate and concrete proportioning methods will be given.

A concrete laboratory was maintained at Hunters Point. Preliminary research tests were made of the aggregates and of aggregate, mortar, and

* Cons. Concrete Technologist (Kitts & Tuthill), San Francisco, Calif.

concrete mixtures, varying the grading, fineness modulus, and consistency for two cement contents (1.5 and 2.0 bbl.) to determine the strength, density, water-cement ratio, weight, cement-space ratio, filler-voids relations, etc. One man-hour of laboratory control was maintained for 10 cu. yd. of concrete. Routine tests of aggregates as received, were made to determine specific gravity, density, fineness modulus, grading, moisture content, bulking, absorption, and content of silt, organic matter, and inferior particles. Calculation and re-adjustment of proportions of aggregates and water were made as required by the natural variation in the grading, density, moisture, absorption, and bulking of the individual aggregates as received. A laboratory duplication was made of each change of proportions to check the physical characteristics expected for that grading, cement, and water content. Routine slump, wash, and compression tests were made for each pour.

Aggregate Test Methods.—The new features of the procedure of testing aggregates are the "weight-volumetric" method of measurement, the regular use of a full container, and the co-ordination of measurements. The procedure is as follows:

(*a*) Weigh the water, filling container.
(*b*) Weigh the loose, moist aggregate, filling container.
(*c*) Dry the aggregate and weigh it.
(*d*) Make a standard sieve analysis.
(*e*) Make a standard silt test.
(*f*) Make a standard test for organic matter.
(*g*) Weigh the dry-rodded aggregate, filling container.
(*h*) Weigh the dry-rodded aggregate, inundated.
(*i*) Allow this to stand 3 hours, then drain, blot, and weigh it.

Having determined these values by measurement, the following may be computed:

(*j*) The apparent density of the dry-rodded aggregate $= \frac{(a) - (h) + (g)}{(a)}$.

(*k*) Apparent specific gravity of the aggregate particles $= \frac{(g)}{(j) \times (a)}$.

(*l*) Apparent density of loose, moist aggregate $= \frac{(c) \times (j)}{(g)}$.

(*m*) Weight of moisture to weight of dry aggregate $= \frac{(b) - (c)}{(c)}$.

(*n*) Bulking of dry-rodded volume by moisture and loose measurement $= \frac{(g)}{(c)}$.

(*o*) Volume of moisture to dry-rodded volume $= \frac{(g) \times [(b) - (c)]}{(c) \times (a)}$.

(*p*) Weight of dry-rodded aggregate per cubic foot $= 62.4 \times (k) \times (j)$.

(*q*) Weight of water absorbed to weight of dry aggregate used $= \frac{(i) - (g)}{(g)}$.

(*s*) Volume of water absorbed by apparent volume of aggregate $= \frac{(k) \times [(i) - (g)]}{(g)}$.

(*t*) Relation of proportion of moisture by volume of dry-rodded aggregate to proportion by weight in the corresponding loose, moist aggregate $= \frac{(o)}{(m)} = \frac{(a)}{(g)}$.

Measurement (*a*) provides a check for the standard container and also makes it possible to use a bucket or other suitable vessel of unknown volume in case of need.

TABLE 2.—TYPICAL CHARACTERISTICS OF SAND AND ROCK.

Factors.	Sand.	Rock.
Apparent density (dry-rodded)	0.663	0.576
Apparent specific gravity	2.654	2.695
Moisture content (by absolute volume)*	0.136	0.014
Absorption (by absolute volume)	0.04	0.025
Bulking of dry-rodded volume	1.245	1.069
Fineness modulus	3.136	7.31
Discolored rock (by absolute volume)	.....	0.094
Inferior rock (by absolute volume)	.....	0.014

* "Absolute volume," as used herein, denotes "apparent volume," as generally termed by concrete physicists. It is the volume within the surfaces of the particles in the cases of aggregates and cement, and the volume at normal temperature in the case of water.

Results of Aggregate Tests.—A fine and coarse sand were tested before each delivery, by assistants stationed at the point of origin, and pre-mixed by the dealer in proportions determined by the assistants. Two sizes of rock were pre-mixed in the same manner. The materials varied somewhat in physical character with each delivery. Typical physical characteristics of the sand and rock, as received at Hunters Point, are as given in Table 2.

Typical gradings of the aggregates, as received, are as shown in Table 3.

TABLE 3.—TYPICAL CONCRETE AGGREGATE.

Sieve No.	Percentage Retained.	
	Sand.	Rock.
100	97.9	100.0
48	82.6	99.9
28	54.7	99.8
14	43.2	99.6
8	29.2	99.5
4	6.0	97.6
⅜-in.	0.0	86.0
¾-in.	....	47.6
1½-in.	....	0.0
Fineness modulus	3.14	7.30

New Method of Grading Aggregates.—An algebraic method of grading three or more sizes of aggregates for practical uniformity of any fineness modulus, as used on this work, should be of considerable interest to those familiar with the graphical method of determining proportions for uniform grading as developed by the late William Barnard Fuller, M. Am. Soc. C. E.*

* "The Laws of Proportioning Concrete," *Transactions*, Am. Soc. C. E., Vol. LIX (December, 1907), p. 67.

The new method combines the fineness modulus principle with the grading equation, $r = 1 - \left(\frac{d}{D}\right)^n$, in which, r is the proportion (by absolute volume) retained by a given screen opening of d in., D is the maximum size of aggregate, and n is an exponent. A grading of any desired fineness modulus is possible by controlling the value of n. The practical values of n vary from 0.45 to 0.60, increasing with the cement content and fineness modulus. When n is 0.5, the curve is a parabola corresponding to Fuller's theoretical grading. From this curve, he deducted a weight of sand equal to the weight of cement used, thus increasing the fineness modulus with the cement content, but losing a measure of uniformity.

The writer's method is to select the maximum desirable fineness modulus consistent with character of aggregate, cement content, and workability, and then find the corresponding value of n.

The desirable fineness modulus for Class "A" concrete was found in the preliminary tests to be 6.00. The grading equation for fineness modulus = 5.99 and $D = 1\frac{1}{2}$ in., is,

$$r = 1 - \left(\frac{d}{1.5}\right)^{0.56}$$

and the grading is as listed in Table 4.

TABLE 4.—MATERIAL GRADED FOR CLASS "A" CONCRETE.

Sieve No.	Square opening, inches in diameter.	Proportions Retained by Absolute Volume.	
		Mixed aggregate, r.	Sand, $\frac{(r - r')}{(1 - r')}$.
100	0.0059	0.9550	0.8556
50	0.0117	0.9340	0.7882
30	0.0234	0.9032	0.6998
16	0.0469	0.8564	0.5892
8	0.0937	0.7884	0.3209
(4)	0.187	0.6884*	0.0000
$\frac{3}{8}$-in.	0.375	0.5399	.....
$\frac{3}{4}$-in.	0.75	0.3217	.....
$1\frac{1}{2}$-in.	1.5	0.0000	.....
Fineness modulus.....	.....	5.99	3.19

* The value of r' is 0.6884 and is the proportion of coarse aggregate.

The fineness modulus of the coarse aggregate is determined by proportion to be,

$$\frac{5.99 - 3.19\,(1 - r')}{r'} = 7.25$$

The fine and the coarse aggregates were accordingly pre-mixed for the ideal fineness moduli of 3.19 and 7.25, respectively, the dealer accomplishing this generally with satisfactory accuracy.

New Basis of Concrete Proportioning.—An (a) absolute volume of mixed aggregate composed of a uniform grading of particles (by diameters and

absolute volumes as in Table 4), with (b) an absolute volume of cement to a unit volume of concrete and (c) an absolute volume of mixing water to a unit volume of cement, was used as an exact basis of mixture. For example, a typical basis of mixture for Class "A" concrete (assuming 1 cu. yd. of concrete) is as follows:

Absolute volume of cement.....$2 \times 4 \times 0.485 = 3.88$ cu. ft.
Mixing water .....$2 \times 4 \times 0.80 = 6.40$ " "
Absolute volume of aggregate $\left(\text{graded to } r = 1 - \left(\frac{d}{1.5}\right)^{0.56}\right)$

$$\frac{27}{0.97} - 3.88 \text{ (cement)} - 6.40 \text{ (water)} = 17.56 \text{ " "}$$

(The determined yield of concrete in place is 0.97 of the sum of the absolute volumes of aggregate, cement, and water).

With aggregates as given in Tables 2 and 3, the absolute volumes of fine and coarse aggregates are determined from the fineness moduli and the total volume of aggregates as follows:

$$17.56 \times \left(\frac{7.3 - 5.99}{7.3 - 3.14}\right) = 5.53 \text{ cu. ft. of sand}$$

and,

$$17.56 - 5.53 = 12.03 \text{ cu. ft. of rock}$$

The mixing water, correcting for moisture content and absorption of the individual aggregates, is:

$$6.40 - 5.53\ (0.136 - 0.04) - 12.03\ (0.014 - 0.025) = 6.00 \text{ cu. ft.}$$

The proportions, measuring aggregates by loose-moist volume, are:

	Loose volume.
Cement	2 bbl.
Added water.....	6.00 cu. ft.
Sand, $\left(\frac{5.53}{0.663}\right) \times 1.245 =$	10.4 " "
Rock, $\left(\frac{12.03}{0.576}\right) \times 1.069 =$	22.3 " "

It should be kept in mind that these calculated proportions are correct only as long as the characteristics of the aggregates remain as given and that about 1 man-hour of laboratory control for every 10 cu. yd. is necessary to keep up with the natural variation of the aggregates.

The characteristics of the usual Class "A" and Class "B" mixes are given in Table 5.

TABLE 5.—CHARACTERISTICS OF CLASS "A" AND CLASS "B" MIXTURES.

Factors.	Class "A".	Class "B".
Fineness modulus	5.9 to 6.1	5.8 to 6.0
Grading, n	0.54 to 0.58	0.52 to 0.56
Barrels of cement per cubic yard	2.00	1.50
Water-cement ratio	0.80 to 0.84	0.90 to 0.94
True mix, cement to dry-rodded aggregate	1 : 3.07	1 : 4.32
Density	0.83 to 0.87	0.82 to 0.86

The usual proportions for 1 cu. yd. were:

	Class "A".	Class "B".
Cement, in sacks	8	6
Sand, loose moist, in cubic feet	11.4	21.8
Rock, loose moist, in cubic feet	21.8	23.1

Compression Tests.—A sample of four to six compression specimens was taken of each pour. Where six specimens were taken, two were turned over to the contractor for independent testing. The specimens were tested at 2, 3, 7, and 28 days and some at other ages.

In view of the erratic nature of compression tests and the law of probability and error in measurement, the probable strength of the samples was determined by taking the average of the following three determinations:

(1) The 28-day strength as measured;

(2) The 28-day strength indicated by the 7-day strength in accordance with the U. S. Bureau of Standards formula, $S_{28} = S_7 + x\sqrt{S_7}$ (x having been determined as 27.7); and,

(3) The 28-day strength as indicated by any two strengths at ages greater or less than 28 days, as determined by the semi-log age rate formula,

$$S = k\ (\log a) + c$$

in which, S is strength at any age, a, in days, and k and c are constants for the particular sample.

Analyses of fifty samples of Class "A" and twenty-three samples of Class "B" concrete show the results given in Table 6.

TABLE 6.—Compressive Strength at 28 Days, in Pounds per Square Inch.

	Class "A", in pounds per square inch.	Class "B", in pounds per square inch.
Minimum	2 965	1 980
Average	3 715	2 725
Maximum	4 415	3 390
90%	3 240 to 4 415	2 240 to 3 390
75%	3 445 " "	2 460 " "
50%	3 775 " "	2 670 " "
25%	3 965 " "	2 950 " "
10%	4 200 " "	3 120 " "

Assuming that the lower 10% of samples is due wholly to the usual adverse errors of compressive tests, the minimum indicated strengths at any age, in any part of the concrete, based on the highest of the lower 10%, are expressed by the following equations:

Class "A",

$$S = 2\,074\ \log\ a + 248$$

Class "B",

$$S = 1\,794\ \log\ a - 356$$

This indicates that the probable minimum strength of the concrete in any part of the shell, at the age of 100 days, is about 4 400 lb. per sq. in. and that in the roadway slab, about 3 200 lb. per sq. in.

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Paper No. 1708

THE VIRGINIAN RAILWAY ELECTRIFICATION*

By George Gibbs,† M. Am. Soc. C. E.

With Discussion by Messrs. W. B. Potter, Marcel Japiot, L. K. Sillcox, I. Öfverholm, Ralph Budd, Guido Semenza, F. H. Shepard, E. R. Hill, Norman Litchfield, Sidney Withington, George J. Ray, John P. Hogan, R. D. N. Simham, and George Gibbs.

Synopsis

The paper aims to present the broad engineering features of an important electric traction installation for long-distance freight service of the heaviest character, in a manner which will make clear to railway engineers generally its essentials in design, construction, and operation. In so doing an attempt has been made to avoid undue length or complication in describing details which are of interest to electrical engineers only.

The physical character and equipment of the Virginian Railway on its heavy grade division, the character and amount of business, and the difficulties experienced in caring for its rapid growth under steam conditions, are first described. The paper then proceeds to outline the methods followed in determining quantities and types of apparatus required to handle the given traffic electrically. A general description of the main elements of an electric traction system is given, and the characteristics of the particular system determined upon as most suitable for the given conditions are then briefly described under the usual sub-divisions of the power supply, the high-tension transmission line, the distribution devices along the railway, and the locomotives.

In this treatment stress is laid on some of the special problems solved, such as the adaptation of the combustion features of the power plant to handle successfully the very large momentary fluctuations in power demand required

* Presented at the meeting of February 1, 1928.

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to start the heavy trains on a grade. Also, the inductive effects set up in paralleling electric circuits normally and during short-circuits are explained, as well as the corrective means adopted. The problems involved in the design of a suitable continuous-contact wire system over the tracks are described fully, and the requirements for a successful catenary system and its supports are illustrated, as matters which are vital to a successful traction system and which present features affecting the work of railway engineers generally.

A short description is given of the inspection and repair facilities provided for the locomotives. Also, the special features necessary for the Company's telephone and telegraph lines along the tracks are noted.

The paper closes with a short description of the manner of conducting the designing of the system throughout, the special methods followed in its construction, and a résumé of operating experience.

Introduction

Within the western slopes of the Appalachians in West Virginia is located a series of bituminous coal deposits of great extent, notably the "Pocahontas", the "New River", and the "Kanawha" fields. Because of the favorable mining conditions, and especially because of the high calorific value and free-burning qualities of the coals from these fields, they are in increasing demand for steaming purposes. They are made available in the markets, both East and West, by three railway systems, the Chesapeake and Ohio, the Norfolk and Western, and The Virginian. Conditions surrounding transportation to the Atlantic seaboard only are, however, dealt with in this paper. These three railways have their tide-water termini at ports around Hampton Roads, Virginia, to which, in 1926, they transported more than 27 000 000 gross tons of coal for vessel loading. Of this amount, 45% was "export" coal, that is, destined to points outside the United States, and the remainder was for domestic consumption. Of the eastbound coal, more than 75% was classed as "low volatile".

The Alleghany Mountains present a complicated topography, characterized by a series of lateral spurs having steep contours and enclosing narrow valleys. Under these conditions favorable railway location is difficult and long adverse grades eastbound are necessary, with many viaducts and tunnels and a large amount of curvature. Furthermore, it is extremely expensive to provide additional running tracks; and to secure room for passing sidings is always a problem and, in places, impossible. Unfortunately, also, these difficult conditions obtain in a section where cars must be distributed to and gathered from mine sidings and spurs.

To compete effectively with Pennsylvania mines, which are at least 100 miles nearer the seaboard and, perhaps, several hundred miles nearer the great coal-consuming centers, these more distant West Virginian coal fields must enjoy relatively low mining cost and be served by very efficiently operated railways to secure minimum transportation costs. One important factor contributing to cheap transportation is the maintenance of an intensive service with trains of the maximum permissible weight moved at a speed which is as high as safety permits. Thus, by quick transference of lading from the cars

to vessels at tide-water, the completion of a prompt "shuttle service" between mines and terminal becomes possible. This character of "turn-around" obviously greatly reduces the equipment required to perform a given service and thus contributes in an important way to the possibility of profit from the low freight rate prevailing in this highly competitive commodity.

Electric Traction on the Norfolk and Western Railway

There is an especially close similarity between the Norfolk and Western and the Virginian Railways, as to grades and alignment, in the coal fields and as far east as Roanoke, Va. (Fig. 1). Each has a "gathering section" west of the mountains and must haul eastbound tonnage over heavy grades to the summit before favorable line conditions are reached. In fact, the two roads lie closely parallel for almost the entire distance from the coal fields to tide-water. The experience of the Norfolk and Western Railway Company, which had built up a large eastbound coal business before the Virginian Railway was constructed, may be briefly referred to therefore as indicating modern methods in the development of economical mass transportation of low-grade freight, especially as bearing upon the advantages of electrification to this end.

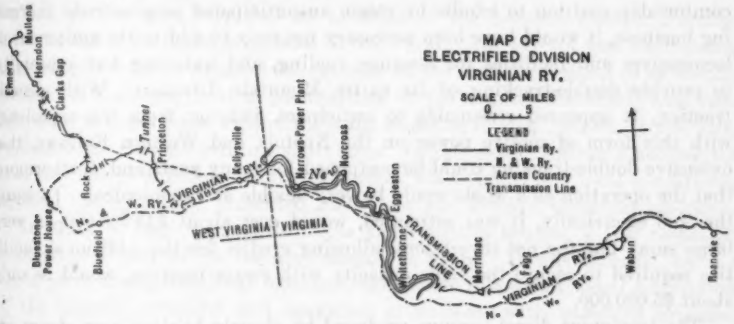


Fig. 1.

The Norfolk and Western Railway Company had used powerful Mallet compound steam locomotives to haul very heavy trains on the mountain grades. The service speed was low, and three locomotives were required per train. In the coal-"gathering" section, comprised within about 30 miles of road immediately west of the mountain summit, the line was double-tracked (except in the single-track ¾-mile Elkhorn Tunnel). With these facilities, however, it was found that service had about reached its maximum possible capacity and economical development. Additional main tracks could only be provided at prohibitive cost through the narrow valleys, where the road was encumbered with mine sidings and coal tipples, and the number of steam locomotives required on this short section was so great as to be a positive hindrance to prompt movements. For example, to handle the coal tonnage of 1914, twenty-four Mallet locomotives were required in as many miles of line, and a round trip—say, 50 miles per day—was about the best average performance which could be secured from each.

The electrification of this "gathering" section of the railway was adopted after an extended investigation as the best solution of operating difficulties and limitations present and future. The installation was completed and put into operation in May, 1915. The new traction method at once effected a reduction in the number of locomotives in service to about one-third that required by steam traction; it doubled, and at times trebled, the daily mileage per locomotive by increasing train speed and by eliminating delays inherent with steam locomotives. Thus, congestion was completely eliminated. During the World War business increased suddenly and very greatly, but notwithstanding this, traffic was handled electrically without difficulty and undue congestion, a feat which the operating officials believe would have been impossible with steam traction.

The Virginian Railway Problem

The Virginian Railway Company had to face similar problems; business had grown constantly until it reached a volume approximating that of its rival, which had more ample track facilities. A comprehensive plan for betterment, therefore, became imperative. In order to place itself in a comfortable position to handle by steam an anticipated progressively increasing business, it would have been necessary not only to add to its equipment of locomotives and facilities for housing, coaling, and watering, but, especially, to provide double-tracking of its entire Mountain Division. With electric traction, it appeared reasonable to anticipate, judging from the experience with this form of motive power on the Norfolk and Western Railway, that extensive double-tracking could be postponed for many years and, furthermore, that the operation as a whole would be very flexible and economical. To equip the line electrically, it was estimated, would cost about $15 000 000—a very large sum—but the net investment, allowing credits for the additional facilities required to secure the same capacity with steam traction, would be only about $5 000 000.

The important direct savings produced by electric haulage over steam are in fuel, time of train crews, and the better utilization of car equipment due to increased speeds, hauling capacity, and reduction in delays. Where, therefore, as in this case, the excess capital fixed charges are not too great, it is reasonable to expect that the improved form of motive power will be directly profitable from the savings and indirectly advisable by reason of its superior flexibility and reliability. After 1½ years of experience with electric haulage on this road, it is gratifying to be able to state that the expectations of the Company have been more than realized in all respects. The direct savings produced have more than met the charges on the net excess investment and the improved operating results have been marked.

Description of the Property.—The Virginian Railway extends from Deepwater, W. Va., at the head of navigation on the Kanawha River, to Sewalls Point, on Hampton Roads, Va., a distance of 443 miles. It was constructed in 1905-07, with a view to obtaining the shortest route and the best possible grades and alignment from the coal fields to tide-water. At present (1928), the average length of haul from mines to tide-water is 405 miles. The road traverses

the Pocahontas, Winding Gulf, and New River Districts of the semi-bituminous (low volatile, or "smokeless") fields and taps a portion of the Kanawha (high volatile) District. It is now serving a total of 110 mines, having a combined potential annual capacity of at least 18 000 000 tons and an estimated recoverable reserve in excess of 1 000 000 000 tons. An additional area having a potential capacity far greater than this figure is tributary to the road.

The Company has two large, modern coal piers at Sewalls Point, designed for the loading of any type of vessel, and each of the two will accommodate four large, or six small, vessels at one time. Each pier is equipped with car-dumpers which can unload a total of 210 cars, or 12 600 tons of coal, per hour.

The railway is single track, except in the 22-mile section on the west slope of the mountains and the 10-mile section west of and approaching the Tide-water Terminal. The aim to provide a coal-carrying railway with grades generally favorable to loaded traffic has been attained, except over the two main mountain ridges intersected within a distance of 134 miles between Elmore, W. Va., and Roanoke (Fig. 1), where topographical conditions necessitate several long heavy grades, producing in effect a "bottle-neck" in the hauling capacity of the entire system.

This section now furnishes an example of the heaviest electric traction development to be found anywhere in the world. The project was undertaken primarily, in order to equalize the capacity of the road throughout so as best to conform to the topographical peculiarities of the country traversed; but, furthermore, to avoid large expenditures for steam equipment, which, it was felt, would become obsolete within a short time. In this paper is given a necessarily brief account of this important installation which under the local conditions, physical and operating, presented many new and difficult technical problems.

In order to keep the length of the paper within bounds, a full technical description of any of its features cannot be given. It was thought, on the whole, more useful to railway men and engineers to furnish a general description of the physical property and operating conditions, with sufficient explanation of the reasons, either operating or technical, for the adoption of the plans and methods followed, as a guide in determining the advisability of heavy electric traction for similar situations elsewhere.

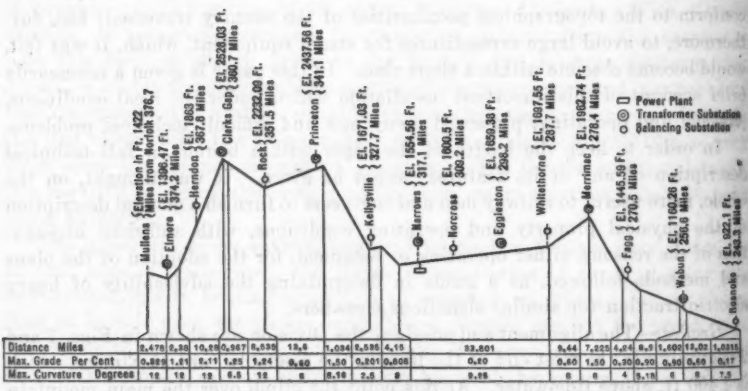
Grades.—The alignment and profile of the Division are shown in Figs. 1 and 2 beginning at the west end of the line, in the assembly yard at Elmore, which is 1 400 ft. above tide-water. At this point the climb over the main mountain ridge starts and continues for 12½ miles on an average 2% compensated grade to an elevation of 2 528 ft. at Clarks Gap, W. Va.; thence over 22 miles of undulating and 0.6% maximum grade to Princeton, W. Va. Near this point the descent occurs and continues for 10.3 miles down a 1.5% grade to Kellysville, W. Va. Thence the line continues over a broken down grade for 6 miles into the valley of the New River, which is followed up stream for 34 miles over a 0.2% adverse grade to Whitethorne, Va. Here, the climb over the Alleghany Ridge begins. This requires about 10 miles of 0.6% grade; it was made a pusher section during steam operation. The mountain summit is reached at Merrimac, Va., from which point the road descends sharply for 7 miles on a

1.5% grade. The remaining distance, 27 miles, to Roanoke is over light grades favorable to loaded movements.

Curvature.—Throughout the entire 134 miles there is a large amount of curvature, varying from 12° maximum and quite generally as sharp as 8°, with the worst curves on the heaviest grades; the mountain section west of Princeton has 60% of its length on such curves. It is evident from Figs. 1 and 2 that both grades and curvature have an important bearing on operating speeds and train loading, and it will be interesting, therefore, to outline the maximum possible service under steam operation.

Steam Haulage Methods

At Elmore, the main assembling point for eastbound tonnage, a train was made up of 5 600 tons trailing weight, summer loading. This was hauled over the heavy grade to Clarks Gap by a road engine of the Mallet type and two Mallet pushers. At that point the pushers were cut off and returned light to Elmore Yard, the train continuing to Princeton with the road engine only. At Princeton, the train was filled to 8 500 tons, summer loading, or 6 000 tons, in winter, for the run to Roanoke. The power for this latter movement was one road engine of the U. S. Mallet type, with a helper engine to assist starting out of Princeton Yard and a helper on the 10-mile grade from Whitethorne to Merrimac.



Distance Miles	2.478	0.38	10.22	0.967	8.530	13.6	1.084	2.535	4.15	53.601			9.44	7.225	4.04	0.5	1.600	13.01	1.0115
Max. Grade Per Cent	0.329	1.21	0.11	1.25	1.24	0.60	1.50	0.201	0.008	0.60			0.60	1.50	0.30	0.90	0.90	0.56	0.17
Max. Curvature Degrees	10	12	12	8.5	12	8	8.16	2.5	8	9.25			8	8	4	3.15	6	8	7.5

Fig. 2.—Track Profile of Electrified Division, Virginian Railway.

The average inclusive speed from Elmore to Princeton, with two stops for water, was about 7 miles per hour. Under best conditions 3½ hours were required for this 36-mile movement. This time, however, was generally exceeded and was often as much as 7 hours, due to bad weather or to poor steaming of the locomotives. The time required for the movement from Princeton to Roanoke was normally from 11 to 12 hours at an inclusive speed of about 8 miles per hour; over-time was frequently required, however. On this part of the run stops were made at four stations for water, at one of which re-coaling and inspection took place.

Westbound the service was wholly an empty-car movement. The trains averaged 2 700 tons and were hauled over the Division by one road engine, without using a pusher on the Merrimac and Kellysville hills. The running time for this light service varied from 12 to 20 hours, depending on the weather, the condition of power, and the congestion on the road.

To conduct this tonnage service the Company used two classes of Mallet steam locomotives, one known as the "U.S." for road haulage over the Division, and another and heavier one known as the "A.E." type, for pushers on the grades. The general characteristics of these locomotives are given in Table 1.

TABLE 1.—Characteristics of Mallet Steam Locomotives Used on the Virginian Railway.

Type.	Wheel arrangement.	Weight of locomotive, in tons.	Weight on drivers, in tons.	Weight of tender, in tons.	Total weight in working order, in tons.	Rated tractive effort, in pounds.
U. S.	2- 8- 8-2	265.5	239	104.5	370	101 300
A. E.	2-10-10-2	343	308.5	107.1	450	147 200

The coal cars used are both hopper and gondola. Of these, 7 100 are of 52 to 55 tons capacity, 500 are of 70 tons, and 2 029 of 110 tons capacity. Overloads of 10% are allowed.

The standard ratings for the trains varied in practice considerably, and were subject to a reduction of 10 to 20% in winter weather and to special reductions at other times, depending on the condition of the steam power, which cannot always be maintained 100% efficient. Furthermore, successful hauling of these extremely heavy trains over a line which is undulating in character, assumes good condition of car draft-rigging and air-brakes, and very careful handling both in starting and stopping. As would naturally be expected, these conditions are not always present and "break-in-twos" were not infrequent.

Due to all the variables mentioned and inherent in steam service, congestion and delays were frequent, especially in bad weather and at times of heavy business, hence provisions for additional facilities became imperative. It was considered that electric traction offered the best and most progressive means of securing these facilities.

Determination of Electric Traction Requirements

Knowing the physical character of the railway, the primary considerations are the power demand at the train and from the system as a whole. These are determined by the weights, speeds, and number of trains in motion at one time.

Train Weight.—Allusion has been made to the very heavy trains operated by steam locomotives, in fact, train weights had been increased nearly to the limit of the allowable stresses at the car couplings in a service conducted over broken grades. At the start of electric operation, therefore, it was believed

best not to attempt a material increase in the standard weight of trains over the low-grade part of the Division, but rather to secure other possible improvements in the operation, especially to shorten the running time, to eliminate pushers east of Clarks Gap, and to handle reliably a fixed train maximum, substantially regardless of weather conditions. In effecting these operating improvements, the type of locomotive and the charactertistics of the electric system throughout should be made flexible enough to permit the running of heavier trains if conditions should so demand in the future.

The train make-up fixed upon as most suitable, all things considered, was 6 000 tons trailing weight up the hill from the assembly yard at Elmore, handled with a head engine and a pusher, this train to be filled out to 9 000 tons at Clarks Gap and handled by a head engine only for the entire 116-mile run to Roanoke. In this way the pusher formerly required on the Alleghany hill from Whitethorne to Merrimac with steam service was eliminated. This train weight and motive-power assignment effects a practical balance between locomotive adhesive requirements on the grades and the electric motor capacity needed for the long sustained runs, and does not increase car draft-rigging stresses beyond those which have been demonstrated as allowable under the physical conditions obtaining on this railway.

Speeds.—It has been stated that it was especially desired to shorten as much as possible the running time between Elmore and Roanoke, which by steam locomotives required frequently as many as 20 hours. Experience on the Norfolk and Western Railway demonstrated that it was undesirable to permit fluctuating speeds which in places might become excessive; in fact, that two standard speeds, reliably maintained, best suited a difficult track profile and alignment. On that road a speed of 14 miles per hour on the heavy grades, where complicated by excessive curvature, and 28 miles per hour on the low grade and light curvature portions of the line have proved to be suitable. In the light of this experience, and because the physical conditions on the two railways are practically the same, these standard speeds were adopted in the electrification of the Virginian Railway.

It should be noted, as will be made clear in describing the design of the locomotives, that the running speeds just mentioned are fixed. They depend only on the motor combinations and do not vary with the grades or loads. Fig. 3 shows the speed assignments adopted for an eastbound run over the Division with a loaded train. It will be seen that about 47% of the distance is covered at a speed of 28 miles per hour and the remainder, which comprises the pusher and the regenerating grades, is run at the relatively low speed of 14 miles per hour.

This set-up results in a running time of 7¾ hours over the 131 miles between Elmore and Roanoke; in practice, however, this may be extended to from 9 to 11 hours, depending on the time taken for filling out the train at Clarks Gap, and for inspection at Whitethorne, which latter is the only other regular stopping place on the Division. It was predicted that under favorable conditions, if not in general practice, it would be possible for an electric locomotive to make a round trip over the Division (262 miles) in 24 hours. This, in fact,

was done frequently during the winter of 1926, and the average mileage per day secured per locomotive in service, in November of that year, was 180. This may be rightly regarded as a fine performance, when it is considered that approximately one-half the number of locomotives are used in pusher and short-run hill services.

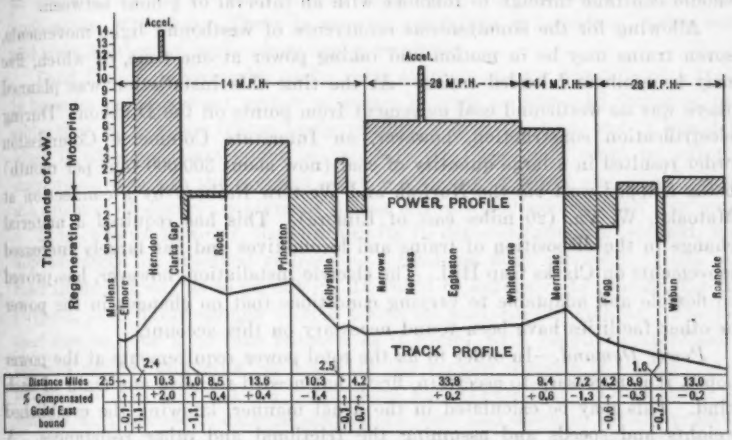


FIG. 3.—ROANOKE-MULLENS ELECTRIFICATION, VIRGINIAN RAILWAY COMPANY. POWER DATA, EASTBOUND.

Yearly Tonnage.—The railway handled 8 306 000 net tons of coal eastbound in 1925, which was an increase of 10% over that handled in 1924. In 1926 traffic took a sudden jump, due, perhaps, to unusual conditions (such as, coal mining strikes in England), and 11 500 000 tons were moved. Assuming, however, a regular and normal yearly increase in business, it was felt that the initial electric equipment should be capable of handling comfortably a total of 12 500 000 tons eastbound per year, and that the plant should be susceptible of ready enlargement to take care of a progressively increasing traffic. The ratio of average to maximum day movement is about 60 per cent. Therefore, the eastbound tonnage on a maximum day becomes 57 000, or a gross of about 80 000 tons of cars and lading. In addition, manifest and local trains and the returning light trains must be moved. A summation of all this tonnage means a total of 110 000 to be moved on a maximum day. In fact, this quantity was actually handled on certain days during 1926.

Number of Trains.—In order to meet the yearly and daily traffic demands outlined, it was estimated that for a heavy day movement fourteen 6 000-ton trains must be hauled up the heavy grade from Elmore in 24 hours. At Clarks Gap these would be combined into nine 9 000-ton trains to be hauled through to Roanoke. In addition, provision must be made for one local freight per day in each direction and a certain amount of switching duty. The empty cars are returned in nine trains of 3 000 tons each, thus balancing car and

locomotive movements eastbound and westbound. Obviously, these trains cannot be spaced evenly over the line as traffic and operating necessities require that a certain amount of "bunching" be allowed. Provision was made, therefore, to permit that two tonnage trains should operate simultaneously on the Clarks Gap Hill, spaced ½ hour apart; also, that two 9 000-ton trains should continue through to Roanoke with an interval of ¾ hour between.

Allowing for the simultaneous occurrence of westbound light movements, seven trains may be in motion and taking power at one time, of which, five may be eastbound loaded trains. At the time this installation was planned there was no westbound coal movement from points on the Division. During electrification construction, however, an Interstate Commerce Commission order resulted in a large quantity of coal (now about 300 000 tons per month) being shipped west *via* the Norfolk and Western Railway by a connection at Matoaka, W. Va. (20 miles east of Elmore). This has required a material change in the disposition of trains and locomotives and has largely increased movements on Clarks Gap Hill. The electric installation, however, has proved so flexible and adaptable to varying conditions that no changes in the power or other facilities have been found necessary on this account.

Power Demand.—In order to fix the total power requirements at the power house, it is necessary to ascertain, first, that needed to move one train of each kind. This may be calculated in the usual manner, knowing the established weights and speeds and assuming the frictional and other resistances. A curve may then be plotted for the train over the entire Division. This curve is applied to the desired train sheet to determine the total power requirements on the assumptions made as to the number and kinds of trains in motion and their spacing. Fig. 3 has been plotted to show graphically an eastbound movement of a loaded train over the line, together with the energy inputs at all points, including arbitrary starts interpolated at critical points to indicate any extraordinary energy requirements. It will be noted that a 6 000-ton train (7 285 tons weight with the two locomotives) on Clarks Gap Hill requires about 14 000 kw. at the train when starting, and the continuous input while running up this 2% grade is about 11 700 kw. The long run up the New River Valley of a 9 000-ton train (9 640 tons with one locomotive) requires about 11 000 kw. during the start and the accelerating period, and 6 500 kw. while running at 28 miles per hour.

At two places substantial amounts of power are regenerated over a considerable time interval, namely, when descending the Kellysville and Merrimac grades, where energy is returned to the contact wire at the rate of about 5 000 kw., the train being held back in this way without the use of brakes and without an appreciable change in speed. The power regenerated during the entire eastbound run is approximately 7 800 kw-hr., which is about 20% of the total energy consumed to propel a loaded train over the Division. A westbound run with an empty train does not regenerate a large amount of power, but the total eastbound and westbound movements, it has been found in actual operation, supply by regeneration more than 8% of the total energy required to operate the road.

It has been found that generally all power regenerated is usefully employed to assist other trains which happen to be in motion, but any surplus current returned is automatically absorbed in special water rheostats at the power plant. Regeneration, furthermore, is very useful in holding smoothly and safely long trains on heavy down grades, and is of material assistance in avoiding "break-in-twos" arising from defective condition or improper application of brakes. There is also an advantage in avoiding undue wear and over-heating of brake shoes and car wheels.

Load Factor.—Obviously, under these extreme energy demands of a train, it becomes an important matter to secure as high a daily load factor as possible, that is, to space trains fairly uniformly over the line, and it would be ideal if seasonal peaks could be avoided. If these ends could be effected, they would result in the greatest possible economy of power and would reduce the amount of electric equipment of the entire installation to the minimum.

It is obvious, however, that uniformity in demand cannot be secured, even approxmately, in practical railroading. Business does not come to the railroad in such a way as to permit it. It would be expensive to provide, for this purpose, yards large enough to hold trains for uniform dispatching. The best that can be done is to take business as it comes, with judgment exercised to prevent undue "bunching" of trains in the various sections of the line to an extent that will overload the power plant and sub-stations. Train sheets for normal and, therefore, extreme conditions were laid down to satisfy practical operation as indicated by experience, always having due regard to the factors producing reasonable plant cost and economy of operation.

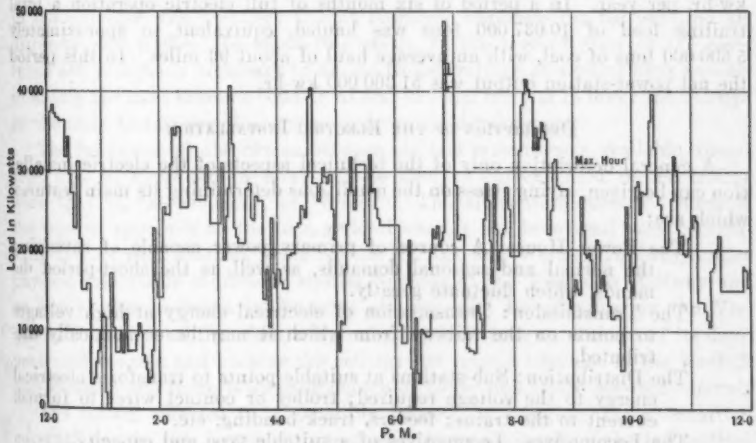


FIG. 4.—TYPICAL HEAVY DAY LOAD, IN KILOWATTS, AT POWER HOUSE.

Daily Load Curve.—A 12-hour curve of loads at the power house, developed from the factors mentioned, for a congested day is given in Fig. 4. It indicates very heavy fluctuations in demand, not only over the hour, but during short intervals. Thus, the maximum 5-min. peaks are of the order of 40 000

to 45 000 kw. and the hourly peaks are about 31 000 kw. Furthermore, the entire load drops at times almost to zero. Under the full electrical operation the peak loads have been substantially as calculated, showing generally a 50% daily load factor, referred to the maximum hour, and at times the load factor has been as high as 65 per cent. As traffic becomes heavier, due to the operation of more trains, the daily average load factor will become greater, but it is estimated that the peak loads will not materially increase.

The loads thus developed at the power house for a normal and for a maximum congested day are shown more clearly in Table 2.

TABLE 2.

Loads.	Normal day, in kilowatts.	Maximum congested day, in kilowatts.
Average load for 24 hours	14 500	20 500
2-hour peak	22 300	29 200
1-hour peak	31 300	31 500
5-min. peak	38 000	43 500

The average daily kilowatt-hour station output was estimated to be about 310 000; the actual output, when handling the volume of business estimated, has averaged 325 000 kw-hr. The yearly station output, of course, depends on the actual tonnage moved. Based on handling a total net coal tonnage of 12 500 000, it was anticipated that the station output would be about 112 000 000 kw-hr. per year. In a period of six months of full electric operation a total trailing load of 10 037 000 tons was hauled, equivalent to approximately 5 500 000 tons of coal, with an average haul of about 96 miles. In this period the net power-station output was 51 300 000 kw-hr.

Description of the Electric Installation

A general description only of the technical aspects of the electric installation can be given, laying stress on the conditions determining its main features, which are:

The Power House: A source of primary power capable of satisfying the normal and seasonal demands, as well as the short-period demands, which fluctuate greatly.

The Transmission: Transmission of electrical energy at high voltages to points on the railway from which it may be economically distributed.

The Distribution: Sub-stations at suitable points to transform electrical energy to the voltage required; trolley or contact wires to furnish current to the trains; feeders, track bonding, etc.

The Locomotives: Locomotives of a suitable type and capacity.

All these features must be designed so as to conduct service successfully irrespective of weather or other variable conditions and in a manner consistent with low installation, operation, and maintenance costs. Furthermore, the system must be such that additions can be readily made to it as traffic grows, and that improvements in apparatus, due to advancement in the

art, may be incorporated without radical and expensive changes in the installation as a whole.

Practical railway operation requires that there shall always be enough equipment provided and power available to permit both normal and congested movements, regardless of the peak loads produced at any point, that is, that traffic movement shall not be subordinated to equipment limitations. Therefore, in determining the quantities of apparatus, this general principle was followed.

The successful development of heavy electric traction for railways is due primarily to advances in the art, which permitted great amounts of power to be transmitted over long distances. As electrical energy is the product of pressure and volume of current, it follows that the higher the pressure the smaller the current required to transmit a given amount of energy, hence the use of very high voltages for power distribution. In early electric traction installations, the distances involved were comparatively short; therefore, moderately high transmission voltages were sufficient for economical results. The distribution voltage in the feeder and contact systems could also be low without unduly increasing the cost of the conductors.

As the distance to be covered increased, transmission voltages were raised for economic reasons, but a more important and difficult problem had to be solved in devising a suitable current-collecting system for the trains. The steel rail formerly used as a contact member necessarily was replaced by an over-head contact wire in which high voltages could be used. Furthermore, new types of motors were developed to utilize high voltages. Logically, the ultimate aim is to find a general system that will permit unrestricted voltages in the collector wire (trolley) to suit economic requirements for any and all distances and traffic. To effect this result, in the present state of the electrical art, it appears necessary to fix the trolley voltage at a point high enough to satisfy the most extreme conditions and in some manner to lower the voltage on the train to suit practical motor and control development.

The fundamental distinction between electric systems now available, therefore, may be expressed in terms of the kind of current used in the trolley wire, that is, "alternating", or "direct". The alternating-current system is the nearest approach so far to a general one; it has been used very successfully for the past fourteen years on the Norfolk and Western Railway, under physical and traffic conditions similar to those of the Virginian Railway and on many other railways in America and abroad. Its adoption on the Virginian Railway was, therefore, a logical procedure, both on account of its demonstrated success, and because the railway lies closely adjacent to the Norfolk and Western and there is an interchange of power and traffic between the two.

The system referred to is generally termed the "single-phase alternating-current", but in this instance three-phase motors are used on the locomotives to obtain certain advantages of this type for the specified haulage conditions, as will be explained later. Single-phase alternating-current, or even direct-current, motors, however, can be used on the locomotives without changes in other parts of the traction system, should it be desired to do so in the future in order to meet changed business conditions.

The main elements and special features of the Virginian electrification will be described, beginning with the source of the power.

The Power Plant

Source of Power.—It is sometimes assumed that an important consideration in determining upon a railway electrification is that advantage can be taken of cheap hydro-electric power. Unfortunately, in the Eastern United States, available water powers are few, and such as exist are not generally near the consumer, and are often not dependable in capacity, so they must be assisted by some form of fuel-burning power plant. Where, therefore, as in the case of the Virginian Railway, high-grade coal is close at hand, a steam plant is suitable and economical, all things considered. It is a reliable means of supply at all times and can be placed so as to meet the needs of the Railway Company.

As an alternative to the construction of a railway power plant for the traction supply, examination was made of the possibility of purchasing power from an existing commercial plant. If power could be secured in this way at a cost not materially in excess of the generating cost of the Company's own plant plus fixed charges, interest, depreciation, and taxes, it would be advantageous, of course, to purchase power and thus save an investment of $4 000 000 in a new plant.

Four different possibilities were examined, but only one commercial power company was prepared to submit a definite proposition and this was so disappointing as to cost, other terms, and conditions, that it was impossible to give it further consideration. As a matter of fact, the load conditions already outlined are so exceptional that it is doubtful whether any existing local company engaged in supplying electric energy to the general public could afford to take on a load of this character, except by devoting special generators and transmission circuits to the railway supply, and thus being under a special handicap in large fixed charges.

Doubtless, there is a field for commercial power companies, especially large networks, in the supply of power for heavy railway traction, but the terms and conditions must meet the ordinary and extraordinary loads of the railway and in order to compete such companies must be satisfied with a low margin of profit, as low power cost is depended upon to furnish the largest direct return on the investment in a railway electrification.

Plant Location.—It will be noted from Fig. 1 that the railway is located in the valley of the New River along which it follows for many miles. This stream varies greatly in volume of flow, but contains at all times an ample supply of water for boiler-feed and condensing purposes. The U. S. Government records for the past 40 years show a water-stage variation of 30 ft. in this river, with maximum floods in 1870 and 1916.

A careful survey was made to determine a suitable power-house site, in view of these high and low-water conditions, at a place where the construction would not restrict the flood prism. Furthermore, it was important to select a site near an established town where employees could be housed. All

conditions were best met at the Town of Narrows, Va., 74 miles west of Roanoke. There, the river valley broadens, and a bend occurs, forming a wide and level plot alongside of which the railway runs at a suitable elevation for connections and sidings to deliver coal and materials to the plant.

General Character.—The plant was designed to insure continuity of service under widely fluctuating loads. Good operating economy was aimed at, but refinements were not included to the extent of unduly complicating apparatus and increasing maintenance difficulties. This seemed to be a conservative procedure, as the cost of fuel is low compared with that found at commercial plants more remote from the coal fields, and as the plant is situated at a considerable distance from skilled labor markets.

There was one consideration which gave the designers concern, namely, the unprecedently great spread between maximum and minimum short-period demands on the steam-generating features of the plant. At certain times, the load at the power house drops off practically to zero (see Fig. 4). At others, there may be only one train on the line taking power and, at still others, there may be four trains, or more. As the instantaneous increments of load may be of the order of 14 000 kw., it is obvious that unusually severe requirements are imposed on the boilers in the production of steam in suitable volume at substantially constant pressure. This same problem had been faced to a lesser degree in the power plant of the Norfolk and Western Railway where the undesirable limitations of the ordinary form of boiler-stoking had been demonstrated.

Therefore, the much more severe load conditions on the Virginian plant appeared to make a change in firing practice desirable and, perhaps, essential. Liquid or gaseous fuel would be ideal, but powdered coal fuel, which has the burning quality of a gas, appeared to offer the most practical solution of the difficulties. The success of powdered fuel had been demonstrated in a number of plants under a variety of conditions elsewhere, thus its use here appeared not wholly experimental. Incidentally, it promised the highest obtainable fuel economy, but its ease of regulation was the main factor determining its use for the Virginian installation.

It is gratifying to be able to state that in the practical operation of this plant for more than 1½ years, the wisdom of using powdered coal has been demonstrated as producing entirely manageable boiler firing under the normal and extreme conditions imposed. The station coal rate has averaged less than ¾ lb. per kw-hr. of net output, or about 14½% efficiency, which, considering the character of the load, is a satisfactory showing. The installed capacity of the power plant was set at 48 000 kw., 1-hour rating, divided into four generating units and five boilers, designed to provide a spare for each.

Building and Foundations.—The local rock is limestone of variable quality, so bedded as to be subject to erosion by underground waters. As the concentrated column loadings were necessarily heavy, special care was taken to explore the site thoroughly by borings to determine sub-surface conditions. Considerable variation in the quality of the rock was found and, unfortunately, the disintegrated portions occurred at certain heavily loaded points.

No cavities were found below sub-grade, but it was thought advisable to establish the low unit pressure of 3 tons per sq. ft. for the heavily loaded stack footings.

In order to establish a reasonable height for the main floor of the building to permit of ready access to the plant, and at the same time to provide economical conditions in the supply of condensing water, the elevation of the basement floor was fixed 30 ft. below high-water mark. This necessitated very substantial walls and floors under the turbine room to withstand the hydrostatic head during flood stages. As the displacement at the basement produced an uplift under these conditions in excess of the total weight of the construction above, it was necessary to insure against any possible movement at such times. This was done by providing a reinforced basement floor slab having concrete ribs between the piers of the turbine foundations, and by anchoring these piers with rods, 1½ in. in diameter, grouted into the rock. Of these anchors, 134 were used. To insure further against any uplift movement, vents were provided in the concrete slab to relieve any pressure which might build up a hydrostatic head.

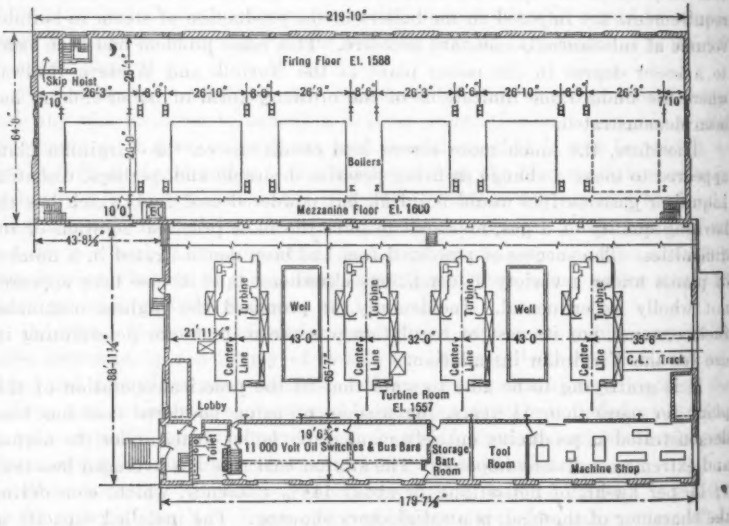


FIG. 5.—FLOOR PLAN, TURBINE ROOM AND FIRING FLOOR, NARROWS POWER PLANT, VIRGINIAN RAILWAY.

The general character and arrangement of the building is shown in the plan and cross-section, Figs. 5 and 6, and in the photograph, Fig. 8. The dimensions and other data of general interest are given in Table 3. The building walls are of brick and are self-supporting; the basement walls are of reinforced concrete. Interior steel framework carries the floors, crane, stacks, and the boiler-room apparatus.

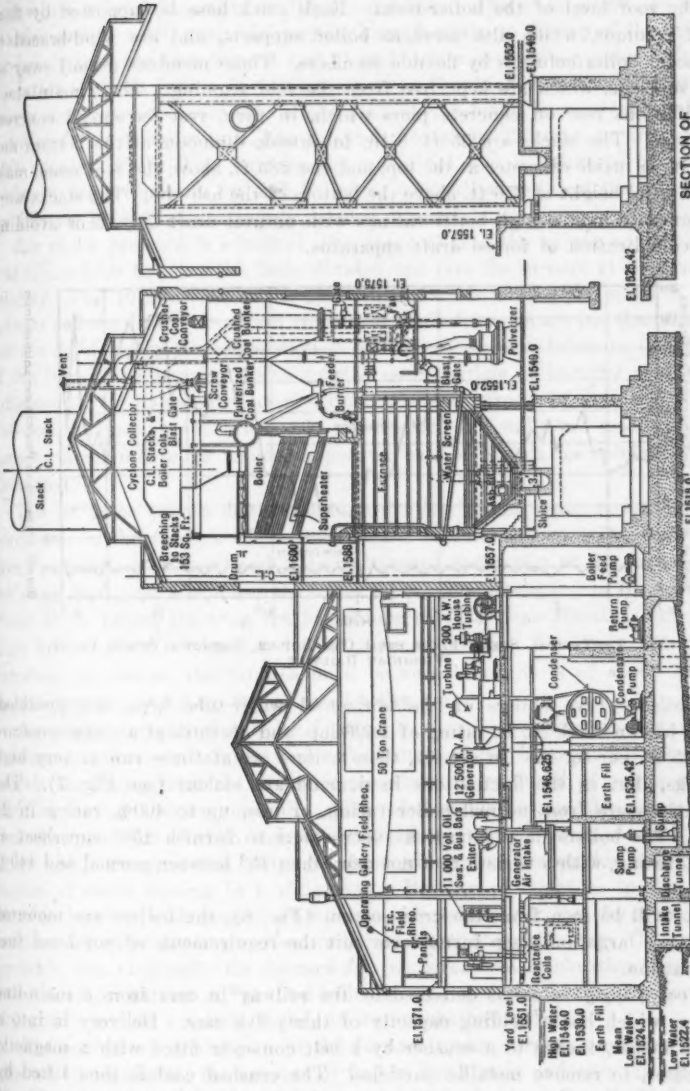


FIG. 6.—PLAN AND CROSS-SECTIONS, NARROWS POWER PLANT, VIRGINIAN RAILWAY.

Stacks.—The two stacks (Fig. 8) are very high, and are constructed of reinforced concrete, supported on, and directly bonded into, concrete mats at the roof level of the boiler-room. Each stack base is supported by four steel columns, which also serve as boiler supports, and are wind-braced to adjacent boiler columns by flexible members. These members permit sway in the supports, which are kept free from the roof framing. These main stack foundations rest on concrete piers which, in turn, rest on spread concrete footings. The stacks are 25 ft. 8 in. in outside diameter at the bottom and 16½ ft. in inside diameter at the top, and rise 285 ft. above the roof mats, making a total height of 376 ft. above the bottom of the ash-pits. The stacks were designed to handle high boiler ratings with natural draft only, thus avoiding the complication of forced draft apparatus.

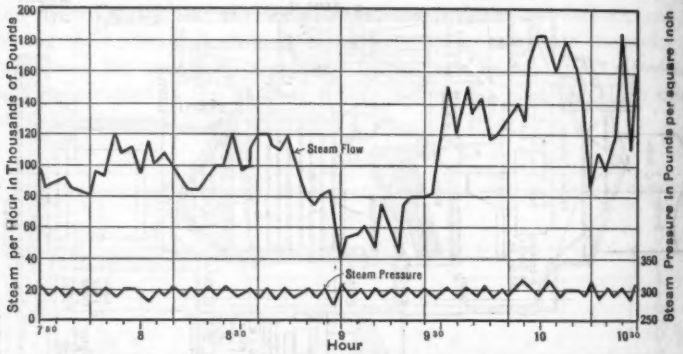


FIG. 7.—TYPICAL STEAM FLOW FROM ONE BOILER, NARROWS POWER PLANT, VIRGINIAN RAILWAY.

Boilers.—Five boilers of the horizontal water-tube type, are provided. Each has an A. S. M. E. rating of 1 520 h.p. and operates at a steam pressure of 325 lb. per sq. in. In service, these boilers are at times run at very high ratings, that is, the fluctuations in demand are violent (see Fig. 7). The variations are from normal boiler rating, or less, up to 400% rating in 2½ min. The boilers have interdeck superheaters to furnish 150° superheat at 160% rating, with a variation of not more than 15° between normal and 440% rating.

As will be seen from the cross-section (Fig. 6), the boilers are mounted over very large and deep furnaces to suit the requirements of powdered fuel combustion.

Coal Supply.—Coal is delivered by the railway in cars from a main-line siding, which has a holding capacity of thirty-five cars. Delivery is into a track hopper, thence to a crusher by a belt conveyor fitted with a magnetic separator, to remove metallic particles. The crushed coal is then lifted by a skip-hoist into bins in the boiler-house, having a total storage capacity of 900 tons. There is also a yard storage space for 2 000 tons, to and from which coal is readily piled and reclaimed by mechanical means.

Pulverized Fuel.—The equipment used to provide powdered fuel takes the crushed coal from bunkers in the power-house building by gravity through spouts into pulverizing mills on the ground floor. The mills grind the coal to a size passing a 150 mesh; delivery is through a pipe to the top of the building by air pressure (see Figs. 6 and 9) and into a cyclone separator. From the latter it drops into a horizontal conveyor in which a screw operates to deliver the pulverized coal into bunkers above the firing floor. One bunker, provided for each boiler unit, has a capacity of about 130 tons, or about one day's supply. From these bunkers the pulverized coal is delivered by screw feeders to ten jet burners for each boiler, so mounted as to inject the fuel into the furnace in divided and nearly vertical streams.

Air under pressure is admitted with the coal into the burner nozzles and acts as a vehicle to carry the finely divided fuel into the furnace at the proper velocity. Fig. 10 is a view of the mechanism for controlling the speed of the burners and the draft. A certain amount of air is drawn in around the nozzles, but the main supply for combustion is introduced through inlets in the walls of the furnace so arranged as to furnish each particle of burning coal with sufficient air for its complete combustion. For the purpose of avoiding excessive temperatures in the brickwork, the furnace side-walls are made hollow, forming a space through which the greater part of the air for combustion is circulated.

The jet flame travels downward toward a horizontal water screen in the lower zone of the furnace where its injection velocity is lost. The flameway then doubles back on itself and is drawn upward at the rear of the furnace by the stack draft over and across the boiler tubes. The jets travel in the furnace about 50 ft. before reaching the boiler-tube surface, thus allowing sufficient time for complete combustion in the furnace proper. To secure this long flameway, of course, the furnace must be very large; it is of about 14 000 cu. ft. contents, or 9.2 cu. ft. per rated boiler h.p.

A certain amount of tubular heating surface is provided in the water screens near the rear wall and across the bottom of the furnace; this serves to protect the lining from the excessive temperatures produced at points where the fuel jet impinges on the walls. The tubes at the bottom also act to prevent the formation of slag, and cause the ash to take the form of small particles which are projected into a pit, from which they are carried away by a stream of water flowing in a sluice-way. Because of doubt as to the most suitable location and amount of furnace side tubing for this form of combustion, a limited heating surface only was supplied in this way. It appears probable that eventually the furnace for powdered coal combustion may contain the major portion of the total boiler-heating surface; that is, this form of firing may lead to a radical change in water-tube boiler design.

The control of combustion in the furnace under varying requirements is effected by regulating the amount of fuel to the burner and also the air supply. Furthermore, individual burners can be "cut in or out" as desired. The combustion is controlled partly automatically and partly by hand. Each boiler is equipped with a control box which contains a system of gearing and

switches to vary the speed of the fuel feed and the movement of the dampers to admit air to the furnace. This apparatus (Fig. 10) is, in turn, controlled from a master switch on the central panel board on which are mounted temperature, pressure, and draft gauges.

The variation of steam pressure in the boiler header actuates a master switch, which regulates movements in the individual control boxes at the boilers. Push-button switches are also provided on the panel to permit of "cutting in and out" individual jets and the adjustment of air and fuel by hand, as required. Thus, the amount of fuel in the furnace at any time can be made to follow closely the plant load conditions and, therefore, the variation in steam pressure can be more uniformly maintained than is possible where there is a large amount of partly consumed solid fuel in the furnace.

As illustrating what has been accomplished in actual practice with this form of firing, Fig. 7 gives a record of the steam-flow meter chart on one of the boilers. The upper line shows that steam requirements varied from about normal to 400% of rating. The lower line gives the record of pressure in the saturated steam header, which fluctuates about 25 lb. under extreme conditions. This is a very satisfactory showing. In practice, it is seldom that the boiler safety valves open on account of excessive rise in pressure.

Steam Turbines.—Four turbo-generators are provided. These have cast steel bed-plates mounted on box girders filled with concrete and supported on concrete piers, thoroughly cross-braced. This heavy monolithic structure is designed to eliminate vibration. Each turbine is furnished with its own condenser (mounted underneath), of the "surface" type, having 20 000 sq. ft. of tube cooling surface. The water supply, amounting to about 20 000 gal. per min. for each condenser, is obtained through an intake tunnel at the upstream side of the power house and discharged through an outflow tunnel down stream.

Each turbine is direct-connected to a generator. This machine is wound for 3-phase 25-cycle current at 11 000 volts. It has a continuous capacity of 10 000 kw. at 80% power factor, single-phase rating. Its single-phase rating for 1 hour is 12 500 kw. A view of the turbine room is shown, Fig. 11.

Transformers.—From the generators the current passes through disconnecting switches to the step-up transformers, which are single-phase and raise the potential to 88 000 volts. Thence, it passes through oil-circuit breakers to the high-tension transmission lines, which deliver current to the sub-stations along the railroad. The step-up transformers, lightning protection, and other sub-station apparatus are placed outside the power-plant building (Fig. 8). The transformers are four in number and are oil-immersed and water-cooled. Each is rated at 10 000 kv-a. continuously, or 15 000 kv-a. for 1 hour, and 20 000 kv-a. for 5 min. following a continuous full-load run. The system power factor is about 85 per cent.

Regenerating Rheostats.—As has been mentioned, on heavy down grades at several places along the railway, a descending train automatically returns electrical energy to the distribution system. Should there be other trains in motion elsewhere and drawing current, the energy thus returned assists in

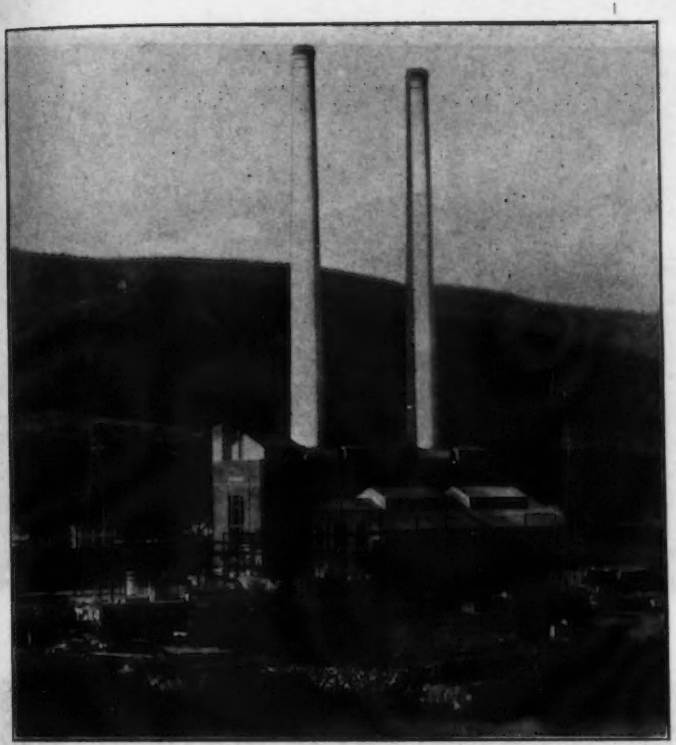
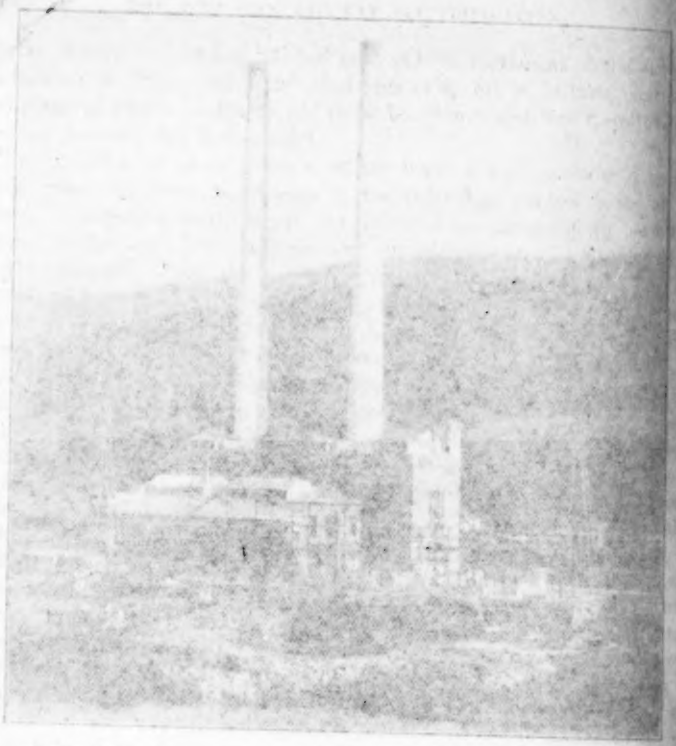


FIG. 8.—GENERAL VIEW OF NARROWS POWER PLANT, INCLUDING SUB-STATION.

FIG. 9.—NARROWS POWER PLANT: VIEW OF BLOWER FOR SUPPLYING AIR TO PULVERIZED FUEL BURNERS.



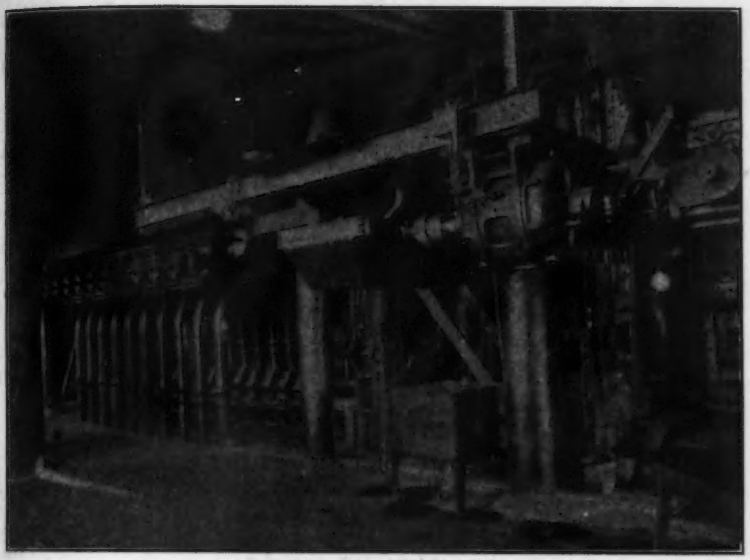


FIG. 10.—MECHANISM FOR CONTROLLING SPEED OF BURNERS AND DRAFT, NARROWS POWER PLANT.

FIG. 11.—VIEW OF TURBINE ROOM, NARROWS POWER PLANT.

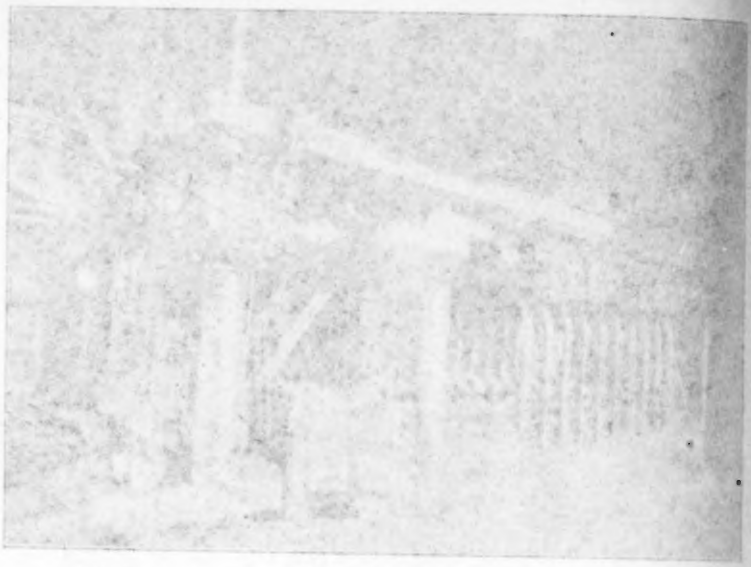


TABLE 3.—Narrows Power Plant. General Data.

(Installed generating capacity (continuous rating), 40 000 kw.; output per year to move 12 500 000 tons of coal, 120 000 000 kw-hr.)

Building dimensions	Boiler room	217 ft. long, 62 ft. wide.
	Turbine room	176 ft. long, 65 ft. wide.
Areas	Boiler room	13 500 sq. ft.
	Square foot per boiler horse power	1.48 sq. ft.
	Square foot per kilowatt generators installed	0.34 sq. ft.
	Turbine room	11 500 sq. ft.
	Square foot per kilowatt generators installed	0.29 sq. ft.
Elevations above sea level	Basement floor	1 525.0 ft.
	Extreme high water	1 549.0 ft.
Stacks	Height above ash-pits	376 ft.
	Height above concrete mat at roof	285 ft.
	Top inside diameter	16 ft.
Fuel used (coal)	Quality	West Virginia low volatile, 13 500 B. t. u.
	Average daily consumption	250 tons (run of mine).
	Capable of handling machinery	85 tons per hour.
	Outside storage on track	30 cars (3 000 tons).
	Outside storage yard	2 000 tons.
	Inside storage, crushed	900 tons.
	Inside storage, pulverized	650 tons.
Boilers	Number installed	Five; space for one additional.
	Rating, each	1 520 b. h. p., A. S. M. E.
	Heating surface, each	15 210 sq. ft.
	Furnace dimensions	26 ft. 10 in. long, 16 ft. 3 in. wide, and 24 ft. 6 in. high.
	Superheater surface, each boiler	1 770 sq. ft.
	Pulverized fuel burners, per boiler	10
Turbines	Number installed	Four; space for one additional.
	Type	Parallel flow, impulse reaction.
	Speed	1 500 rev. per min.
	Operation	275 lb. steam pressure, 150° superheat, vacuum, 29 in.
	Maximum economy	12.48 lb. of steam per kw-hr.
Generators	Continuous rating, each	10 000-kw., 25-cycle, 3-phase, 11 000-volt.
	Exciters, direct-connected	125-kw., 250-volt, direct-current.
Condensers	Number installed	Four.
	Type	Surface; two-pass.
	Tube surface, each	20 000 sq. ft.
	Condensation water required, each	20 000 gal. per min.
Circulating pumps	Number installed	Four.
	Type	Centrifugal, with direct-connected variable speed, alternating-current motors.
	Capacity, each	20 000 gal. per min.
Condensate pumps	Number installed	Four.
	Type	Centrifugal, with direct-connected alternating-current motor.
	Capacity, each	300 000 lb. per hour, against 135-ft. head.
Boiler feed pumps	Number installed	Three.
	Type	6 in., four-stage, steam, turbine-driven.
	Capacity	650 gal. per min.
Feed-water heaters (open)	Number installed	Two.
	Type	Metering.
	Capacity, each	300 000 lb. per hour.
Feed-water heaters (bleeder)	Number installed	Four.
	Type	Closed.
	Capacity, each	200 000 lb. per hour.
House service water supply	Filters	Three, vertical pressure type.
	Capacity of each filter	350 gal. per min.
	Filter wash tank	One.
	Capacity of tank	7 500 gal.
Auxiliary power turbo-generator	Turbine	One unit, operating 6 000 rev. per min.
	Generator, gear-connected	300-kw., 440-volt, 3-phase, 750 rev. per min.
	Exciter, direct-connected to generator	100 kw., 250-volt, 750 rev. per min., direct-current.

TABLE 3.—(*Continued.*)

Air compressor, vacuum cleaner system	Installed capacity	One, 150 cu. ft., motor-driven, 100-lb. pressure.
	Type	One, vacuum producer, inlets on several boiler-room floors.
Crane	Installed capacity	50 tons with 10-ton auxiliary hoist.
	Type	One electric traveling, in turbine room.

propelling them. Should there, however, be current returned in excess of the amount required by other trains, this excess must be absorbed in some manner to prevent overspeeding the generators at the power house. For this purpose, water rheostats are provided. These are single-pole electrodes placed in the water at the mouth of the discharge tunnel. They cut in automatically and quietly absorb the surplus energy. It is found in practice that these rheostats seldom come into action, because over a long railway line generally a train is in motion to make use of all regenerated energy.

As the foregoing description of the power plant is very brief, additional statistical information is appended in Table 3.

High-Tension Transmission

From the step-up transformers at the power house a main transmission line distributes current to the sub-stations along the railway. Such transmission circuits are generally carried either on the poles or bridges supporting the catenary wire system, or a separate pole line may be located on the right of way. The Virginian Railway is a single-track line, except for 22 miles at the west end. The right of way is narrow, with a great amount of curvature, and is complicated with many tunnels and high viaducts. Moreover, future track changes are probable.

It was decided, therefore, to carry the transmission across country on a separate right of way, planned so as to intersect the railway only at the sub-stations, which are spaced about 20 miles apart. By reason of the rather circuitous alignment of the railway through the mountain passes, it was found possible in this way to secure a direct line transmission 34 miles shorter than the route along the railway right of way. At the westerly end, on the 22-mile double-track section, however, the transmission is carried on the catenary poles. At Matoaka, Va., a tie transmission line 9 miles long was run across country to connect with the Norfolk and Western Railway electric system at Bluestone, Va., thus tying together the power systems of the two companies for use in emergencies, or for the interchange of power.

Towers.—The line traverses a very broken country with many ridges and deep valleys which are heavily wooded. Economy of construction, therefore, required long wire spans. These are, normally, 900 ft. long, but a number are of considerably greater length, the longest being 2 440 ft. Locations for the towers were selected, where possible, upon local ridge crests, thus allowing the desired wire sag with a moderate tower height without encroaching on safe clearances at mid-span.

Square towers of the lattice type were used. These towers are of galvanized steel, and the framing and panel lengths are made suitable for economical transportation and setting up. They were designed to resist the unbalanced stresses caused by two broken wires and the tensions were limited by allowing the wires to slip in the supporting clamps. To the stresses produced under these assumptions was added that caused by a wind pressure of 8 lb. per sq. ft. acting on the projected area of ice-covered conductors plus 12 lb. per sq. ft. on 1½ times the projected area of one tower face. Ice loading of ½ in. radially was assumed as occurring simultaneously with maximum loading and wind pressures. Fig. 12 shows the design of the 88 000-volt suspension towers.

The tower footings consist of stubs and grillages embedded in the ground to provide adequate resistance against uplift, and designed for a low unit value for bearing resistance against settlement, with due allowances for secondary strains, which might develop due to deflections in newly tamped earth back-fill.

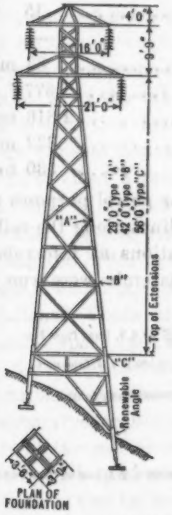


FIG. 12.—PLAN OF 88 000-VOLT SUSPENSION TOWER.

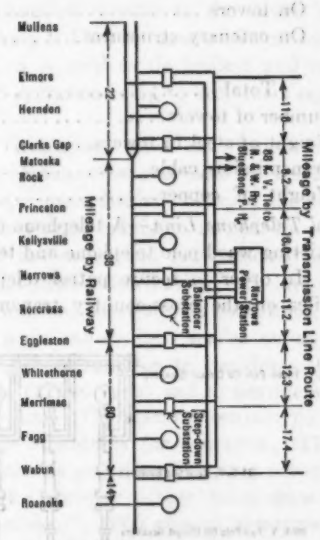


FIG. 13.—DIAGRAM OF POWER DISTRIBUTION SYSTEM.

Circuits.—The towers carry two single-phase 88 000-volt circuits, with the wires, arranged vertically, two on each side. The conductors are of hard-drawn stranded copper, having the following equivalent section:

For the 41 miles of transmission east of Narrows Power-House .....3/0 copper
For the 16 miles westward to Princeton...250 000 cir. mils.
For the 26.5 miles from Princeton to Elmore.....4/0 copper

On the upper cross-arms, the wires are spaced 16 ft. apart horizontally and, to prevent the possibility of the lower wires whipping and striking the upper

ones, those on the lower cross-arm are normally spaced 21 ft. apart horizontally. This separation is particularly necessary at the summits of the mountains where during the winter there is danger of "frozen fog", which is a crystalline mass, often of large diameter and of about the consistency of snow, forming on the conductors. The part of this mass struck by sunlight melts quickly and falls off, whereas the part in the shade remains, thus producing an unbalanced load and causing the cable to whip upward. This would require great vertical separation in order to avoid short circuits if the two cables on one side of the tower were in the same vertical plane.

The conductors are suspended from a string of seven porcelain disk insulators, with clamps so designed that slippage will occur at about 1 000-lb. pull in case of a broken conductor. Anchor towers are provided at crossings over railroads and also where necessary to take dead-end stresses. Miscellaneous data for the transmission line are as follows:

Total length:	
On towers	68 miles
On catenary structures	15 "
Total	83 miles
Number of towers	377
Weight of steel in towers	1 816 tons
Transmission cable	332 miles
Weight of copper	530 tons

Patrol Telephone Line.—A telephone circuit for patrol purposes was added to the existing wood-pole telephone and telegraph lines along the railway right of way. In order to secure patrol telephone stations at intervals of about three miles on the cross-country transmission, laterals were run from the railway.

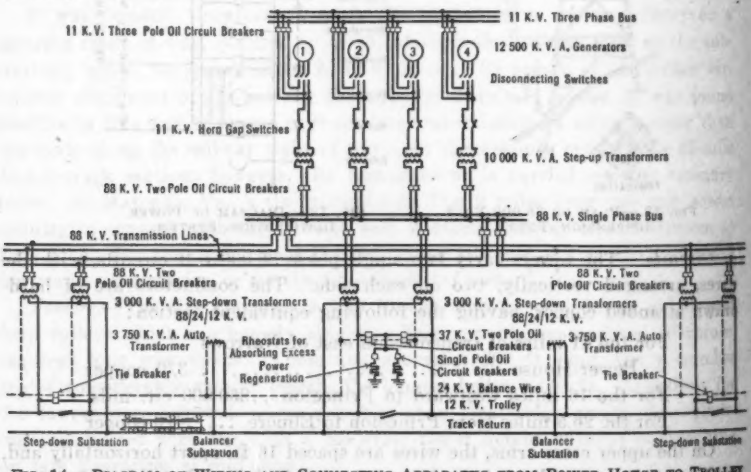


FIG. 14.—DIAGRAM OF WIRING AND CONNECTING APPARATUS FROM POWER HOUSE TO TROLLEY WIRE AND RETURN, VIRGINIAN RAILWAY ELECTRIFICATION.

Distribution

The distributing wiring and apparatus comprehends the various features of the traction system, beginning at the points where the transmission is tapped into the sub-stations. It includes the switching facilities, the transformers, the catenary-supported contact, the balancing wires over the tracks, and also the track-bonding and return conductors.

The general scheme of distribution is shown in Fig. 13 giving the relative location of the power plant and the transformer sub-stations, which are of two classes, "main step-down" and intermediate auto-transformer (or balancer) stations. Fig. 14 shows diagrammatically the complete electrical connections from the power plant to the train.

Attention is especially called to the simplicity of the switching arrangement. At the power house the step-up transformers are semi-permanently connected to the generators and all switching is done at 88 000 volts. At the sub-stations the transformers are connected directly to the trolley and balancing wire, and the principal switching is likewise done at 88 000 volts. One 11 000-volt breaker in each sub-station is used to tie trolleys and balancing wires together in emergencies, normal operation being to keep them separate in each direction from the sub-station.

In this distribution system the "balancing" wire and "balancing transformers" are unusual features. The arrangement was used for the first time on this work to effect a maximum separation distance between the step-down sub-stations and, at the same time, to reduce induction in near-by communication circuits. It was regarded as the safest and most efficient means of accomplishing the latter purpose in an installation having very large concentrated power demand normally and unusual short-circuit conditions.

Sub-Stations.—There are seven main sub-stations spaced on an average about 20 miles apart (see Fig. 13). These function to step-down the 88-kv. current to the trolley working voltage (11 000 volts) and to provide switching and sectionalizing for the catenary lines. The stations contain no rotating or other apparatus requiring regular attendants for operation. The transformers are of the self-cooled type and the switches, lightning arresters, etc., are of types, which need no housing for protection from the weather. Therefore, the stations can be made "out-of-door". The general arrangement may be seen in the view, Fig. 15. All these sub-stations are situated near a telegraph office or signal tower, where an employee is on regular duty and can be called on in emergency to open or close the oil-circuit breakers, under telephone instruction of the Power Director. Statistics regarding these main transformer sub-stations, also the balancing transformer sub-stations, are given in Table 4.

The seven balancing transformer stations are located approximately midway between the main sub-stations, and contain auto-transformers, non-automatic disconnect switches, and lightning arresters only. They have no connection with the 88-kv. transmission, in fact, they cannot be considered sub-stations in the ordinary sense of the word.

TABLE 4.—Data Relative to Transformer Sub-Stations.

	Distance from West End of Division.		
	Miles.	Number of transformers.	Total transformer capacity, in kilovolt-amperes.
Main Transformer Sub-Stations:			
Elmore, W. Va.	2.6	3	9 000
Clarks Gap, W. Va.	16.1	3	9 000
Princeton, W. Va.	36.0	2	6 000
Narrows (step-down), Va.	69.6	2	7 500
Eggleston, Va.	78.5	2	6 000
Merrimac, Va.	98.3	2	6 000
Wabun, Va.	120.0	3	9 000
Balancing Transformer Sub-Stations:			
Herndon, W. Va.	9.0	3	11 250
Rock, W. Va.	25.3	1	3 750
Kellysville, W. Va.	49.0	1	3 750
Norcross, Va.	67.6	2	7 500
Whitethorne, Va.	89.0	2	7 500
Fagg, Va.	106.2	1	3 750
Roanoke, Va.	133.0	2	7 500

An important departure from usual practice is in the type of transformer used. It is equipped with a third coil on the low-tension side. This coil furnishes current of double the trolley voltage to a "balancing" wire, which is carried on the catenary poles, and connects all main sub-stations and also the auto-transformers located at seven places along the line, midway between the main and at the easterly end of the contact line. While, in one sense, these auto-transformers are auxiliary sub-stations, their function is, in fact, quite different, as will appear subsequently. Their general arrangement and appearance is shown in Fig. 16.

Balancing System.—The essential elements of this novel form of distribution are shown in Figs. 13 and 14. They are: The three-coil transformers in the main sub-stations; the auto-transformer sub-stations; and the connecting "balancing" wire.

The three-coil step-down transformer has one of its secondary windings connected between the trolley wire and the track, and furnishes the propulsion current at 12 000 volts (11 000 volts, nominal); the other secondary is connected between the balance wire and the track and carries balancing current at 24 000 volts. The contact wire, the track, and the balancing wire thus form a three-wire system having a connecting feeder of double the trolley voltage. The reasons for the higher feeder voltage are: First, higher auxiliary transmission voltage of about 36 000, resulting in economy of copper and permitting greater through feed between sub-stations; second, the contemplated ultimate use of 24 000 volts at the trolley so that the feeder voltage, which is of opposite polarity, will counterbalance the static effect; and, third, the better balancing of track currents to points of load, thereby reducing the inductive effect on adjacent communication circuits.

Inductive Effects.—The conditions producing induction in paralleling circuits must be referred to at some length in order to make the functions of the corrective devices clear. Energy is transferred through space from one

FIG. 15.—TYPICAL MAIN TRANSFORMER SUB-STATION.

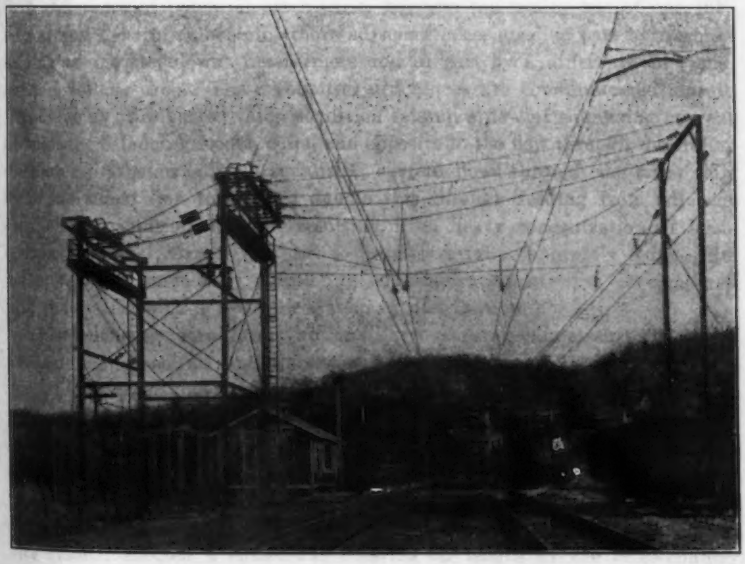


FIG. 16.—TYPICAL BALANCING TRANSFORMER SUB-STATION.

alternating-current circuit to another, due to changes in voltage or amperage of currents in either circuit, and is called "induction". The magnitude of this effect depends on a number of factors and conditions and is common to all forms of power transmission and distribution in varying degree. It is especially great under the condition of derangement of the power lines which causes grounds or short circuits. The extension of the application of powerful electric currents in transportation and for the manifold every-day uses of the public must continue, as must also the use of feeble currents in "communication" systems. Neither should be allowed to exclude the essentials for the successful application of the other, which means that correctives against interference may have to be applied in both systems to insure satisfactory operation.

Various methods, or a combination of methods, may be utilized to reduce or mitigate troubles from induction in communication circuits. Of course, the most effective way is to secure considerable separation between the feeble current line and a paralleling power line. Where a sufficient separation cannot be obtained, a practical method for reducing induction from traction currents is by some form of three-wire traction distribution, of which the latest development for heavy railway work is the balancing wire system mentioned. In stating this, of course, it is understood that the communication line must be the highest modern standard and, in addition, that special devices may be required in it to obtain the fullest immunity, even when all practical correctives have been applied to the power line.

A difference of potential often exists between the trolley bus of one sub-station and the trolley buses of those adjacent, occasioned in part by unequal loading of the step-down transformers and in part by a difference in transmission voltage drop. These potential differences are communicated also to the balancing wire buses, which condition sets up a flow of equalizing current through the balancing feeder, equal and opposite to the flow through the trolley conductor. Substantially no equalizing current flows through the track rails from one sub-station to another due to this through feeding from adjacent ones. This condition, whether resulting from heavy concentration of loads or from abnormal current flow due to insulation failures, greatly reduces the inductive effect on closely paralleling communication circuits as compared with that usual with a two-wire system of distribution.

This balancing wire forms an auxiliary transmission circuit to utilize the installed capacities at adjacent sub-stations more fully than otherwise would be possible; and it effects, also, an improvement in voltage regulation. At the far end of the railway electrification it permitted the installation of an auto-transformer without the necessity of extending the high-tension transmission eleven miles to that point. The somewhat devious route of the railroad, occasioned by the topography of the country, and the wide spacing (20 miles) between main sub-stations, secured by the adoption of this balancing wire system, effected a considerable saving in length of the cross-country transmission by means of long direct-line runs. This would not have been possible with the ordinary and close sub-station spacing; it would have been

necessary to run the line nearly parallel with and close to the railway so as to tie in at the more frequent points.

Sectionalization.—Novel methods were adopted in the circuit switching for this installation. In the event of a short circuit in a trolley section supplied by two sub-stations, the breaker supplying the heavier current caused by the "short" usually opens first, leaving a stub-end short-circuit flow of current from the adjacent sub-station until the current is ruptured at that point. In the meantime severe induction would occur in a two-wire distribution system. In a balancing wire system, however, by using automatic circuit breakers only on the high-tension side of the transformers, the opening of one high-tension breaker and the bus tie breaker leaves both secondary coils of the step-down transformer still connected across the trolley and the balancing wire, which now acts as an auto-transformer, so that stub-end induction effects are avoided at all times.

High-tension breakers, actuated by directional relays, connected in both trolley and feeder conductors on either side of the tie breakers, are so arranged that only the breaker directly feeding the "short" will open, leaving the adjacent section still energized. The bus tie breakers open automatically in case of short circuit. They are also arranged to re-close automatically on the re-energizing of the line from either end, thus establishing through feed to the load from both directions.

The Catenary

The term, "catenary", refers to the form of vertical curve which the messenger cable takes when subjected to a uniform loading between supports; but in connection with an electric traction system the word is generally used to designate broadly the contact devices and supports for such systems over the railway tracks. The contact system is, perhaps, the most vitally important part of a traction installation. It is the means of getting current to the train at any place and under any conditions. As it must be adapted to widely varying physical conditions, and must function properly under all, much ingenuity has been exercised by designers in the development of an effective, durable, and sightly structure and one which can be applied at a reasonable first cost.

The Virginian installation illustrates what has been accomplished in these respects by years of experience with such work. The supporting structures are poles or columns, in single piece; brackets or cross-wires supporting the catenary wire proper; and back-guying to provide anchorage against cross-stresses. The use of high voltage in the trolley wire makes it possible to keep down the dead weight of the system by reducing the size and number of conductors to be supported. In the design of this wire, two elements are of first importance, namely, the contact wire must have sufficient current-carrying capacity to supply all demands and it must provide a sparkless collection of current. To accomplish the latter, the contact wire must be maintained in a fairly level position over the track and be supported so that it has a substantially uniform "contact quality" or "softness". Practical experience in the

design of such a system is of special value in successfully dealing with this difficult problem of sparkless current collection.

The sparking tendency at a contact may be minimized by reducing the amount (amperes) of current collected. While opinions differ as to the maximum quantity collectable, there can be no doubt that the smaller the current the better will be the collection without excessive pantograph pressure on the wire. Logically, therefore, for all reasons, the trolley current voltage should be made as high as practicable. Consequently, the efforts of designers of modern electric traction installations have been directed toward raising the trolley voltage to the highest possible value.

Amount of Current to Be Collected.—The Virginian Railway locomotives consist of three cabs, each of which is equipped with a pantograph having its sliding surface in the form of two parallel thin metal shoes. Each shoe is intended to collect its proportionate share of current. In service the locomotive is run with two pantographs in operation which results in four sliding contacts functioning on the trolley wire. The voltage in the wire is normally 11 000 and the current required at this voltage, to accelerate a train, is from 1 000 to 1 200 amperes per locomotive, or about 300 amperes per contact. During normal running about 150 amperes per contact are collected.

Experience on other railways indicates that 300 amperes can be collected from a single trolley wire with a comfortable margin for the variables occasioned by the usual grade of maintenance of a complicated wire system of the kind. However, in the case of the Virginian Railway, to provide for the possibility that in the future it may become desirable to raise the trolley voltage, the transformers and switches in the sub-stations and on the locomotives have been so constructed that the working voltage may be made 22 000 by simple and inexpensive changes in connections.

Size of Wires.—The contact and supporting wires, over the heavy demand portions of the railway, were designed to carry 1 500 amperes for 10 min. on a hot day without suffering a loss of tension through over-heating sufficiently to result in a badly sagging line. On the light grade portions, the system was designed to carry 850 amperes for 10 min., under similar conditions. The distribution of this carrying capacity is as follows:

The trolley wire is a 3/0 grooved wire of copper tin bronze, drawn to a tensile strength of 60 000 lb. per sq. in. and characterized by its resistance to wear, toughness, and ability to withstand high tensions continuously without permanent stretch. Its conductivity is 65% of that of copper. Above, and parallel to, the trolley wire is an auxiliary messenger of hard drawn copper. This wire acts as an electrical feeder in addition to assisting to secure a soft contact line; finally, the main messenger (catenary cable) has considerable electrical conducting capacity. This messenger is a composite cable, having a stranded core of special composition bronze wires of 125 000 lb. per sq. in. tensile strength and surrounded by a covering of hard-drawn copper wires. No special feeder cables were required in addition to the essential wires of the catenary construction described. In a sense, however, the "balancing" wire is a feeder, but it was used for another main purpose, as has been mentioned.

Suspension Method.—The single trolley, or contact, wire is carried from an auxiliary wire by clips; this latter wire is, in turn, suspended from the main messenger by wire hangers, clipped to the auxiliary. The messenger cable is hung through a string of insulators of the suspension type to a bracket carried from a side pole, in single-track construction (Fig. 17), or from a cross-catenary wire carried by two poles, one on either side of the tracks, in cases of multiple tracks (Fig. 18). This, in general, is normal tangent construction. Special methods of suspension, however, are required to insure a "soft" contact line on curves and through overhead wire switches and section breaks.

Materials.—Reference has already been made to the non-ferrous composition of the wires. In addition, all trolley hangers and clips are of bronze. This non-ferrous wire system produces greater variations in tension, due to temperature changes, than a steel messenger cable, and, consequently, a greater variation in the height of the trolley wire. The tension of the latter, however, was made sufficiently high so that in summer it will not become flabby and thus produce an undesirable variation in contact conditions; and, on the other hand, so that tensions shall be within the elastic limit of the materials in extremely cold weather. No mechanical tensioning devices are used or required under any condition, as in a properly designed system the temperature stresses are relieved by the elasticity of the wires, by the means of carrying them, and by the elasticity of the supporting poles and cross-wires. The frequent occurrence of curves, also, materially assists in distributing stresses.

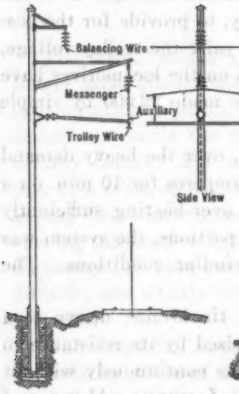


FIG. 17.—TYPICAL TANGENT BRACKET.

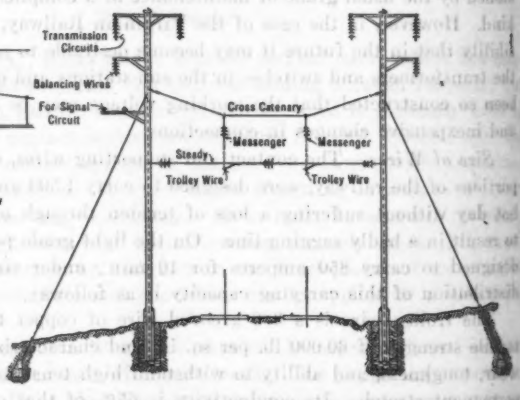


FIG. 18.—TYPICAL TANGENT CROSS-CATENARY BRIDGE.

As it is strung the trolley wire is given a tension of 3 000 lb. at a temperature of 60° Fahr. This tension is expected to increase to approximately 4 800 lb. at a temperature of 20° below zero. The messenger is given a sag of 5½ ft. in a 300-ft. span at a temperature of 60° Fahr.

Curves and Special Work.—Two methods of hanging the trolley wire from the messenger are in general use in America for curved tracks. One is the "chord" construction, in which, by a series of pull-offs, the trolley and messenger are kept approximately over the center of the track in a series of

chords. This form of construction is somewhat unsightly, as the catenary bridges or pull-off poles must be placed comparatively close together to avoid too great a departure of the wires from the track curve. Chord construction was used only in the yards on the Virginian Railway.

The other method—the one used on the main line—is the "curved catenary" construction, in which the trolley wire follows the center line of the track fairly closely around the curve. This is accomplished by the use of inclined hangers, the messenger being offset the proper amount at its supports, as predetermined by calculation. This form permits the supporting structures to be spaced farther apart than in case of chord construction, and the wire system presents a very neat appearance. Where the track curves are very sharp, "pull-offs" for the trolley wire are used in addition, to permit greater spacing of the supporting structure and to limit the incline of the hangers to an angle which will not cause interference with the pantograph on the locomotive.

The sloping hanger construction produces flexibility by allowing the wires to swing around the messenger cable as a center, and tends to eliminate "hard spots". A "hard spot" may be caused either by the increased inertia of a heavy fitting, inserted locally, or by a cusp or kink in the wire; also, by the application to the contact wire of a force (push or pull) in a vertical direction. Any of these conditions has the effect of interrupting the wave produced by an advancing pantograph, which thus may be made to rebound from the wire. This, of course, results in sparking and burning. In the design of the hardware, or fittings, it is the aim to reduce the concentrated weights as much as possible, also to avoid rigidly braced points, vertical cusps, or sudden changes of direction of the wire, and struts that directly push or bear down on the wire.

Section Breaks.—In order to divide the trolley properly into convenient electrical sections insulated one from the other, to prevent faults from being communicated to the whole system, and to isolate sections when making repairs, some form of separation must be used between adjacent sections, which will permit the sparkless passage of the locomotive pantograph across the break. Wooden sticks have frequently been used for this purpose but they are not entirely satisfactory as insulators, are mechanically uncertain, and are so heavy as to cause "hard spots" in the trolley wire, which tend to cause arcs as the pantograph passes.

To avoid the use of wood and to secure a soft line, "air-gap" section breaks are employed. In these, the two sections of trolley wire are lapped past each other, separated horizontally about 12 in. At the point of transfer the wires gradually rise above the level of the contact line and their tension is finally carried back through porcelain strain insulators at a higher level into the trolley of the next electrical section. The lapped trolleys are suspended from the same messenger by hangers and are kept separated horizontally by bronze deflector rods.

Insulators.—The trolley insulators are of the suspension type and are used in strings of three. They are porcelain disks, 10 in. in diameter, cemented into malleable iron housings. Each unit has a rating of 16 000 volts and the three

together will withstand a dry flash-over test of 190 000 volts. In certain tunnels, on account of limited clearance, it was impossible to use the suspension insulators. Therefore, two-piece insulators of the rigid type, two in series, were utilized. Great care was exercised in the design of the tunnel insulation, because of the importance of insuring reliability in the presence of moisture, smoke, and gases, and because of the difficulty of making renewals under traffic.

Supporting Structures.—In general, the side supports, or poles, for the catenary are spaced from 300 to 320 ft. apart on tangents. On curves, the spans must be shorter, for reasons already given. On very sharp curves, special pull-off poles (see Fig. 19) are used in addition to the supporting poles.

For single-track, and for a portion of the two-track, line, the supporting structure is a single pole carrying a bracket (Fig. 17); the major portion of the installation is of this type. For multiple tracks a one-piece pole, on either side of the track and back-guyed, is used (Fig. 18). In this latter construction no rigid cross-member is required. Both tapered tubular poles or H-columns of a uniform section are suitable as vertical supporting members, but in the case of the Virginian Railway the H-column type was employed because of economy, because it is easily climbed by the use of "climbing skates" and because wire attachments are readily made. The cross-member, which carries the main catenary system, is a wire strand, a form of construction the engineers have developed and used with success on other railway electrifications. The distinctive features of the catenary system described, are the production of a relatively cheap and elastic construction which is as light as is consistent with durability and safety. Incidentally, the design offers the least interference to vision along the tracks from the locomotive. Fig. 20 shows cross-catenary construction on yard tracks.

Poles or columns having continuous surfaces are to be preferred over latticed structure supports, not only from the standpoint of first cost, but also because they present vertical surfaces quickly drained and automatically cleaned by rain, whereas lattice-bar columns tend to collect dirt and moisture, which lead to corrosion. Repainting is also a relatively simple and cheap performance.

Foundations.—Heretofore, it has been common practice to set poles on concrete foundations poured in forms in place. In this installation a novel type of setting was devised which greatly reduced the cost of the foundations and did not require the distribution of concreting materials along the tracks, with the attendant added cost and considerable interference to traffic during construction. It was aimed to approximate the simplicity and cheapness of a wooden telegraph-pole setting. In carrying this out the columns were first served with a reinforced concrete casing on their lower ends. This was done in a yard at Roanoke some weeks in advance of distributing the columns to their sites. The design of this reinforcing and protecting casing is shown in Fig. 21.

The foundation proper was made by digging holes of comparatively small diameter in the desired locations along the track; broken stone was then placed

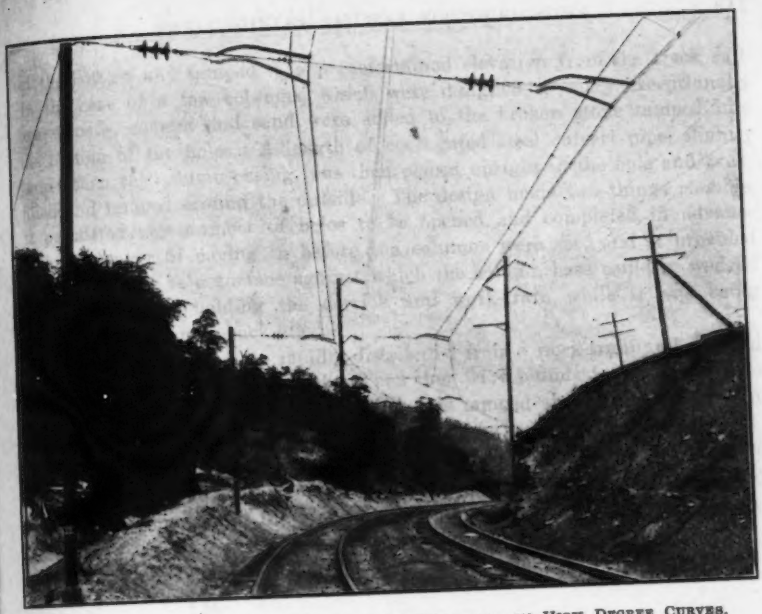


FIG. 19.—TYPICAL TWO-TRACK CATENARY CONSTRUCTION ON HIGH DEGREE CURVES.

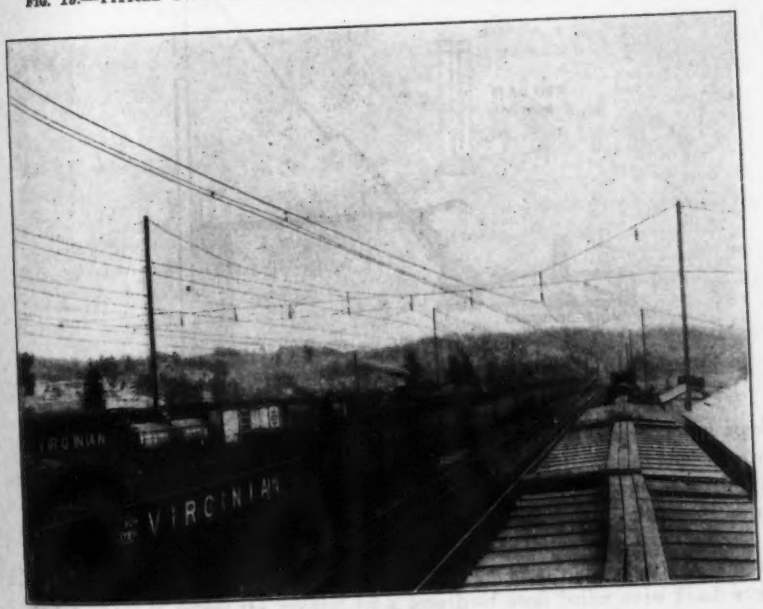


FIG. 20.—CROSS-CATENARY CONSTRUCTION ON YARD TRACKS.

at the bottom and tamped to a predetermined elevation from the track rail. In the case of a few columns, which were designed to carry exceptionally heavy loads, cement and sand were added to the broken stone tamped into the bottom of the holes. A length of corrugated steel culvert pipe, slightly larger than the column casing, was then placed upright in the hole and back-filled and tamped around the outside. The design made two things possible. It permitted any number of holes to be opened and completed in advance without danger of caving in before the columns were set; and it provided the culvert pipe as a surface against which the column base could be wedged and left, without holding the derrick and work-train while it was being plumbed and the hole back-filled.

The cased columns were rapidly distributed from a work-train and dropped in place in the holes by a derrick. Then they were plumbed and the culverts filled with earth and broken stone, which was tamped around the casings. A surprisingly strong bond was thereby obtained between the base and the culvert pipe, as was demonstrated when there was occasion to pull up and re-locate several columns. In such cases, it was found that the column, casing, fill, and culvert pipe came out of the ground as a unit when pulled up by the derrick.

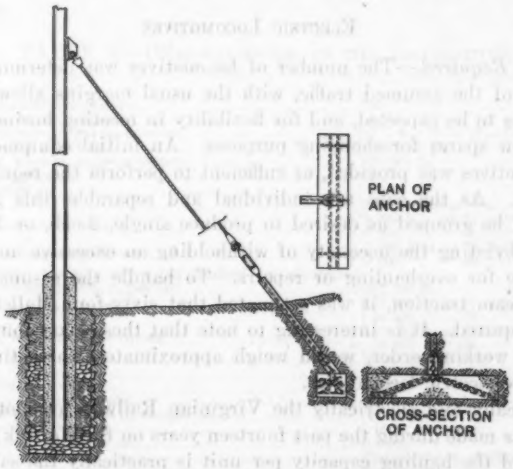


FIG. 21.—GUY AND ANCHOR FOR CATENARY SUPPORTS.

Guys.—The columns are guyed with extra high strength galvanized steel strand attached to anchors set in the ground (see Fig. 21). These anchors are pre-cast rectangular blocks of concrete, 6½ ft. long by 18 in. wide by 14 to 17 in. deep, and weigh from 1 720 to 2 100 lb. each. They are reinforced with longitudinal rods, 1 in. square, and cross-rods, ½ in. square.

The guy strand is connected to the anchor by a steel rod stub which is protected where it enters the ground by a length of steel boiler tube filled with grout. The guy is attached to the anchor by a swivel connection and to the

strand by a turnbuckle. The stresses on these guys and anchors are considerable, amounting at times to 35 000 lb., but experience covering several years (in three other installations) has shown that the construction is an entirely dependable form of pole bracing, where space permits of its use, and that it constitutes a valuable means of lightening and cheapening catenary construction. On tangents, the bracket posts are used without guys; on curves, they are back-guyed.

Track Return.—The means of returning the current from the locomotive to the sub-stations is through the bonded track rails. The running tracks are equipped with both 130 and 100-lb. rails and with lighter rails in sidings and yard tracks. All main running track rails are connected with two bonds per joint. Passing sidings have one bond only per joint; yard tracks are single-bonded, in one rail only. The bonds are No. 1/0 copper with expanded pin terminals, installed under the joint plates. Cross-bonds, each consisting of one No. 4/0 copper wire, are inserted at ½-mile intervals.

At the main sub-stations and balancer transformer sub-stations the track rails are connected to the buses by weather-proof insulated copper cables laid underground in trunking.

Electric Locomotives

Number Required.—The number of locomotives was determined from the train sheet of the assumed traffic, with the usual margins allowed to permit of the delays to be expected, and for flexibility in meeting business variables; also, to allow spares for shopping purposes. An initial equipment of twelve 3-cab locomotives was provided, as sufficient to perform the required schedule comfortably. As the cabs are individual and separable, this gave 36 cabs which could be grouped as desired to produce single, 2-cab, or 3-cab locomotives, and obviating the necessity of withholding an excessive number of cabs from service for overhauling or repairs. To handle the assumed volume of traffic by steam traction, it was estimated that sixty-four Mallet locomotives would be required. It is interesting to note that these sixty-four steam locomotives, in working order, would weigh approximately three times as much as the twelve electric locomotives.

Mechanically and electrically the Virginian Railway locomotive embodies developments made during the past fourteen years on the Norfolk and Western Railway, and the hauling capacity per unit is practically the same as in the case of the latest electric locomotive adopted by the neighboring road. The 3-cab locomotive of the Virginian Railway represents the greatest concentration of tractive effort in one singly controlled machine found in any present-day practice, steam or electric. To secure this, advantage has been taken of the fact that an unlimited number of driving axles can be electrically actuated by control from one point, in the manner familiar in multiple-unit electric passenger-train operation. Great capacity is thus readily secured by semi-permanently connecting together separate frame units, each with its own assemblage of wheels and driving mechanism designed for proper tracking characteristics.

Starting Conditions.—In fixing on a type of locomotive, the difficulty of starting very long and heavy trains on mountain grades must be taken into account. A successful start can be made only in a very gradual manner, by a series of impulses taking up the car-coupling slack in the train progressively, moving very slowly, a few cars at one time, until all are in motion. Otherwise, train "break-in-twos" will result.

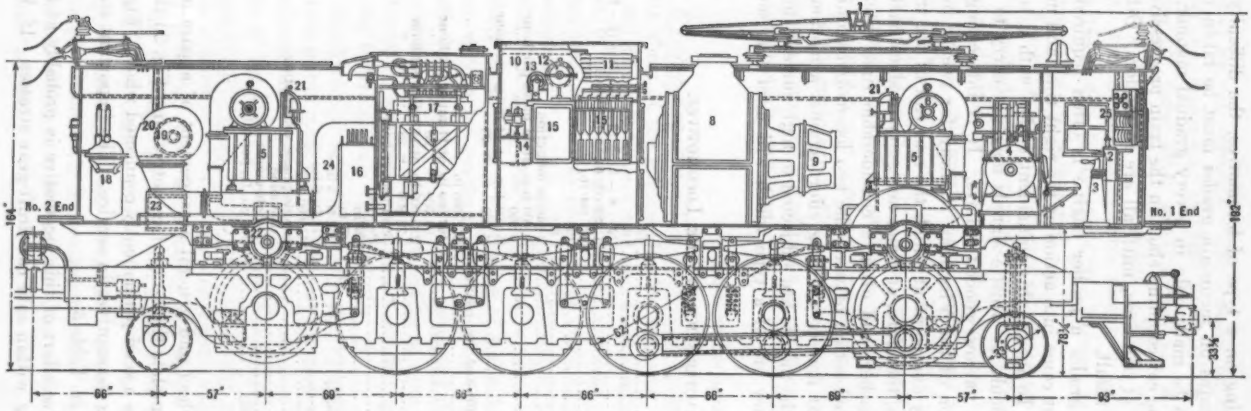
Furthermore, on the heavy grades a pusher engine is often required at the rear of the train, and it must come into action at the right time and must take up its share of the load. To insure this coincident action with a train nearly a mile long is often difficult, as whistle signals may not carry to the rear, especially in crooked and narrow wooded valleys. It is interesting to note that on the westerly portion of the Virginian Railway for twenty miles on the heavy grade, there is only one point at which the rear end of the train can be seen from its head. It is, therefore, at times necessary to slack back to give a "bump" signal to the pusher, and often a considerable time interval may elapse in obtaining concerted action between the two locomotives, during which period the one at the rear may be exerting effort against a motionless train. Under these conditions the motors are subjected to high-current inputs and may be damaged unless they are of a type which will meet this severe requirement.

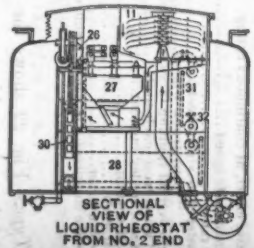
TABLE 5.—Characteristics of the Locomotives.

Number of three-unit locomotives	12
Number of cabs, or units each	3
Classification as to wheel arrangement	(2—8—2) + (2—8—2) + (2—8—2)
Total weight, approximately	1 285 000 lb.
Total weight on drivers, approximately	937 000 lb.
Number of driving axles	12
Number of motors	6
Type	3-phase induction
Total capacity, at 1-hour rating	6 000 h.p. at 14 miles per hour 6 990 " " 28 " " "
Total capacity, continuous rating	5 000 h.p. at 14 miles per hour 6 000 " " 28 " " "
Maximum starting tractive effort (as limited by electrical capacity equivalent to 30% adhesion)	277 500 lb.
Tractive effort, at 1-hour rating	162 000 lb. at 14 miles per hour 94 500 " 28 " " "
Tractive effort, at continuous rating	135 000 lb. at 14 miles per hour 78 800 " " 28 " " "
Maximum safe speed	38 miles per hour
Gear ratio	21 : 100
Diameter of driving wheels	62 in.
" " pony-truck wheels	33 in.
Length over-all (pulling faces of couplers)	152 ft. 3 in.
Rigid wheel-base (each unit)	16 ft. 6 in.
Voltage of trolley current	11 000
Kind of current	Single-phase, alternating
Transformer	Oil-cooled
Phase converter	Synchronous
Air-brake compressor capacity (each unit)	150 cu. ft. per min.

Type.—Fig. 22 shows the wheel and general arrangement of apparatus of one of the units making up a complete three-unit locomotive, and Fig. 23 shows the manner of grouping units in a semi-permanently connected whole. Fig. 24 is a view of one of these electric locomotives in service. The important statistics of the locomotive are given in Table 5.

Tractive Effort.—The tractive effort of this locomotive is probably as high as desirable, in view of keeping within safe car draft-gear stresses. If, how-





1. Pantograph
2. Master Controller
3. Air Brake Valve Pedestal
4. Air Compressor
5. Control Apparatus
6. Traction Motor Blower
7. Traction Motor No. 1
8. Phase Converter
9. Phase Converter Motor
10. Liquid Rheostat
11. Cooling Tower
12. Valve Operating Mechanism
13. Interlock Drum
14. Auxiliary Dump Valve
15. Electrode Compartments
16. Preventive Coil
17. Main Transformer
18. Oil Circuit Breaker
19. Transformer Oil Pump Motor
20. Blower
21. Blower Motor Switch
22. Traction Motor No. 2
23. Radiator for Transformer
24. Air Duct for Rheo. & Prev. Coil
25. Air Gauges & Ammeters
26. Operating Motor
27. Electrode Compartment
28. Electrolyte Tank
29. Rheostat Circulating Pump
30. Main Overflow Valve
31. Fresh Water Storage Tank
32. Electrolyte Tank Filling Valve

FIG. 22.—DIAGRAM OF ONE OF THREE-UNIT ELECTRIC LOCOMOTIVES, SHOWING APPARATUS, VIRGINIAN RAILWAY.

ever, it should be demonstrated in the future that longer and heavier trains can be handled reliably, the tractive effort of the locomotive may be readily increased one-third by adding one more unit. Provisions for this contingency have been made not only in the locomotive, but in the power system throughout.

Drive.—The running gear of each cab comprises four driving and two truck axles (Fig. 23). The drive is through side rods from the jack-shafts, each of which is connected to two pairs of wheels by main and side rods. These two driving couples are not interconnected because it was regarded better practice to avoid excessive rigidity of the system. The rod motion is a purely rotary one and, therefore, can be perfectly balanced by counterweights, thus eliminating the effect of dynamic augment at the track, such as necessarily results from the unbalanced reciprocating parts of the steam locomotive. The jack-shaft gears have spring-cushioned rims, one gear being mounted on each end of the jack-shaft.

The use of a rod drive is advantageous, in allowing maximum length of motor armatures, and hence maximum capacity, also in giving a simple form of connection between parts of the mechanism having relative vertical play, due to the wheel equalization and the springs. Furthermore, coupling of the axles by side rods insures maximum total adhesion for the locomotive as it avoids diminution of tractive effort which occurs with separately geared axles, due to inequality of weight distribution and to weight transfer at low-track rail joints, etc.

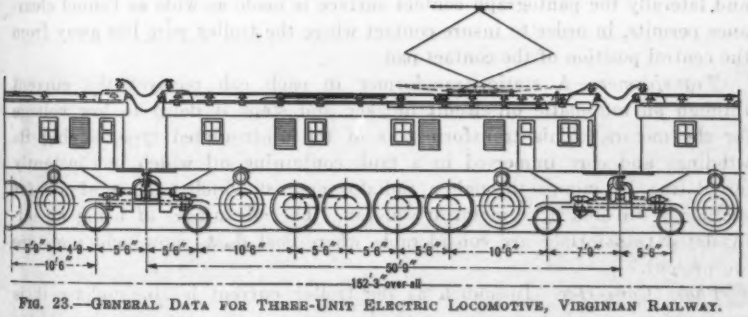


FIG. 23.—GENERAL DATA FOR THREE-UNIT ELECTRIC LOCOMOTIVE, VIRGINIAN RAILWAY.

Motors.—Each locomotive unit has two motors (Fig. 22), one set near each end of its frame and geared to a jack-shaft mounted directly beneath. The motors are 3-phase, having non-variable speed characteristics, that is, they operate at certain set speeds, irrespective of the loading and determined by the combination of motor-pole connections set up. Two standard running speeds were adopted, namely, 14 and 28 miles per hour. The former is used on the heavy grades; the latter on the level and light grades.

It has been found on the Norfolk and Western Railway, in similar service, that these speeds admirably suit the haulage of heavy "drag" freight trains, insuring the maintenance of schedule without excessive speeds over any part of the run. In certain kinds of railway service one inflexible speed

characteristic is objectionable, but in "drag" service where every train is of maximum weight it is advantageous, as the maintenance of absolutely uniform speed, regardless of change of grade, tends to prevent "break-in-twos" due to acceleration and retardation locally within the train itself.

Furthermore, the proper running speed should be accurately gauged and maintained over a road with the grades and curvature found on the Virginian Railway (Fig. 25). When regenerating down grade, the speeds obtainable are about 15 and 30 miles per hour. The change from motoring to regenerating, or *vice versa,* is automatic as the grade changes, without the master controller being thrown to the "off" position. The only action required of the engineman to accomplish the change is a movement of the transformer balancing switch lever.

The three-phase motor is especially well adapted to the difficult train-starting conditions, referred to previously. It is simple and robust in the construction of its revolving element and has no commutator; it is also of large output capacity for a given weight and dimensions.

Current is furnished from the contact wire to the locomotive and controlled through the main electric apparatus. The arrangement and relation of these various parts are shown in Figs. 22, 23, and 24.

Pantograph.—This is a lazy-tong contact device set on the roof of the locomotive cab. It collects current at 11 000 volts from the trolley wire. The mechanism has latitude of motion vertically for the variable wire heights, and laterally the pantograph contact surface is made as wide as tunnel clearance permits, in order to insure contact where the trolley wire lies away from the central position of the contact pan.

Transformer.—A static transformer in each cab receives the current through an automatic oil-circuit breaker and steps it down to low voltage for the motors. This transformer is of the oil-insulated type, having its windings and core immersed in a tank containing oil which is constantly circulating by pumps through a radiator against which a blast of air for cooling is directed by a motor-driven fan. The advantages of using an oil-insulated transformer are considerable where coal dust, snow, and moisture are present.

Phase Converter.—Inasmuch as the trolley current is single-phase it is necessary to provide apparatus on the locomotive to convert it to three-phase for use in the induction motors, hence the term, "split-phase", is sometimes used to describe this type of locomotive. This conversion is effected by means of a rotary "phase-converter", which consists of a synchronous motor with a two-phase wound stator and a wound rotor. One motor stator phase is fed from the transformer and proportional voltage is generated in the other of a phase displaced approximately 90° electrically from the first. One terminal of the generated phase is connected to the mid-point of the transformer secondary, resulting in a "Scott" connection to give three-phase power. The power factor of this three-phase circuit is high (about 95%) by reason of the synchronous type converter used. In an emergency the converter may be operated as a straight induction type.

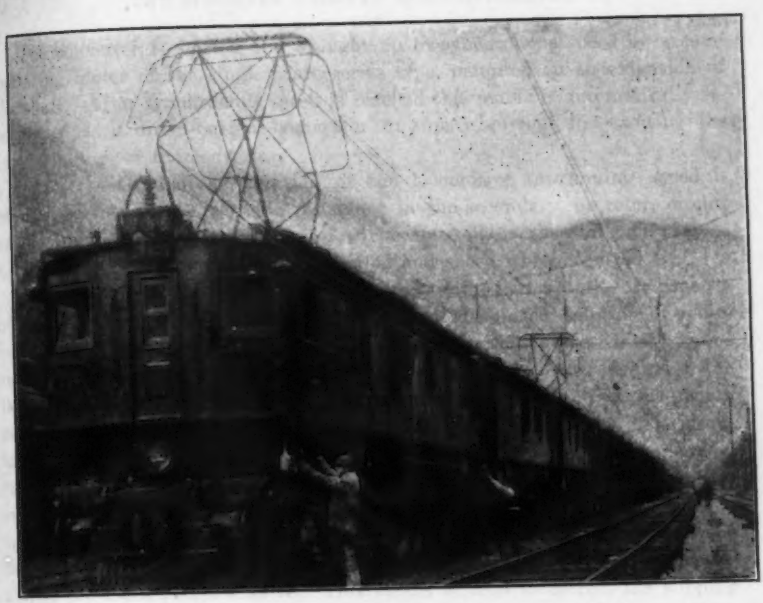


FIG. 24.—ELECTRIC LOCOMOTIVE, VIRGINIAN RAILWAY.

FIG. 25.—EMPTY TRAIN DESCENDING MOUNTAIN GRADE NEAR WEST END OF LINE.

The converter is started and brought up to synchronous speed by a commutating motor of the single-phase series type, mounted on an extension of the shaft. After synchronous speed is reached this motor is automatically reconnected as a direct-current generator to supply current for exciting the rotor windings of the converter.

Rheostat.—Gradual acceleration of the locomotive to running speed is effected by inserting a variable resistance in the secondary, or rotor, circuit of the traction motors by means of a rheostat. This rheostat can also be used to vary the locomotive tractive effort and speed when required.

The rheostat is of the electrolytic type, contained in a tank. It consists of metallic electrodes immersed in an unsaturated aqueous solution of sodic-carbonate. This electrolyte is controlled by an overflow valve which may be raised or lowered by a motor-driven mechanism to vary the height of the liquid around the electrodes and thus to vary the resistance in the motor circuit. As the electrolyte absorbs energy and thus becomes hot, it is circulated by a pump through a cooling tower. In addition to the main overflow valve, which regulates the height of the liquid, two auxiliary valves are provided, one for each electrode compartment. Either of these valves can be opened by a button in the motorman's controller to lower the level of the electrolyte rapidly in the compartment it controls, independently of the position of the main overflow valve. This means is used to correct the slipping of one set of drivers, or to reset the overload relays of one motor without affecting the tractive effort of the other.

Mechanical Construction.—The main backbone of each locomotive unit consists of two cast vanadium-steel side-frames, each in one piece, 42 ft. long, and tied together by end and intermediate castings which form motor supports and cross-ties. Between these ties a cast-steel bed is provided to form a continuous support for the heavy electrical apparatus, such as the phase converter, liquid rheostat, transformer, air compressor, etc., mounted on it. The whole frame assemblage is unusually strong in order to resist bumping and pulling shocks and any abnormal strains such as would be imposed if the locomotive should be lifted with the pedestal binders of the driving axle removed.

The frames and bed-plate carry all the driving mechanism and apparatus independently of the cab, or housing, mounted on it. This arrangement also provides means for supporting the cab rigidly on the substructure without the use of intermediary springs, and makes the cab, in effect, merely a box or shell to cover the apparatus and to carry the pantograph. Hatches are provided in the roof so that all apparatus may be lifted out by the shop traveling crane.

At each end of the frame is a two-wheeled pony truck. It is of the radius bar type with special suspension designed to give the desired resistance characteristics for tracking around curves and to distribute wheel-flange pressures properly between the truck and drivers.

No effort has been made to lighten the mechanical parts in order to obtain high weight efficiency of the locomotive as a whole. It was regarded

of importance, rather, to obtain ruggedness and durability in an extremely severe service.

For ready reference, a short summary of construction and operating data is given in Table 6.

TABLE 6.—CONDENSED DATA.

Dates	Engineering started	June 1, 1923
	Construction started	November 29, 1923
	Regular electric operation commenced between Mullens and Princeton	September 24, 1925
	Regular electric operation commenced over entire Division, Mullens to Roanoke	September 18, 1926
	Note.—Construction work suspended east of Princeton for three months, October to December, 1925, and resumed in January, 1926.	
Right of way	Route miles electrified	134
	Track " "	231
	Maximum grades, percentage	2
	Maximum curvature, degrees	12
	Percentage of line on curves of more than 2 degrees	54
Capacity	Tonnage of coal eastbound for which electrification was designed:	
	Per year	12 500 000
	Per maximum day	57 000
Train weight	Locomotives, cars, and lading on Clarks Gap Hill (two locomotives), in tons	7 285
	East of Princeton (one locomotive), in tons	9 640
	Westbound empty movement (one locomotive), in tons	3 440
Train speeds, running	Percentage of run at 14 miles per hour	53
	Percentage of run at 28 miles per hour	47
Maximum number of trains in motion and taking power	Simultaneous, eastbound	5
	" , westbound	2
	Total taking power at one time	7
	Total number of loaded cars moved eastbound on heavy day	2 200
	Total tonnage moved per heavy day eastbound and westbound (gross)	95 400
	Ton-miles (trailing) per month	160 000 000
Power demand	Starting 7 285-ton train, accelerating to 14 miles per hour on 2% grade, at train, in kilowatts	14 000
	Starting 9 640-ton train, accelerating to 28 miles per hour on 0.2% grade, at train, in kilowatts	11 000
Regeneration	Power, in kilowatts, regenerated down hill by 9 640-ton train	5 000
	Power regenerated in percentage of total consumed in an eastbound run over the Division	20
	Power regenerated in percentage of total power-house output	8

Power-house load	Normal day.	Congested day.
Average load, 24 hours, in kilowatts	14 500	20 500
2-hour peaks, in kilowatts	22 300	29 200
1-hour peaks, in kilowatts	31 300	31 500
5-min. peaks, in kilowatts	38 300	43 500
Total yearly power station output on basis of 12 500 000 tons of coal hauled eastbound, in kilowatt-hours	112 000 000	

INSPECTION AND REPAIR FACILITIES

Locomotive inspection facilities are required at the end of a run. At a turn-around point—such as at Roanoke—where it is undesirable to hold locomotives out of service for a considerable length of time, only minor routine inspection is done. The provisions made for this are: A track pit with housing over it, sanding facilities, and a supply of water for refilling the locomotive control rheostats. All these facilities are conveniently located for quick access by the locomotives.

The main repair shops for locomotives and cars and the Motive Power Department headquarters are at Princeton, 37 miles east of the Division terminus. These shops continue to be required for general equipment repairs for the road, other than for the electrified portion. It was deemed inexpedient to depend on the Princeton plant for all electric locomotive overhauling and repairs, as it was inconveniently located in reference to the end of the electric locomotive runs and the point where they are held for service. Therefore, it was decided to use the Princeton Shops for heavy and class repairs only of the electric equipment, and to build at Mullens a modern and well-equipped plant for general overhauling, minor repairs, and thorough inspection (Fig. 26).

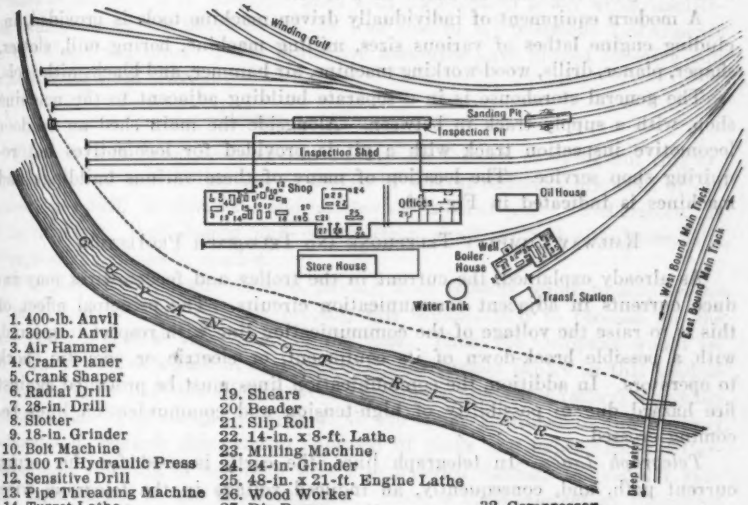


FIG. 26.—LAYOUT OF MULLENS SHOP, VIRGINIAN RAILWAY.

Mullens Shop.—The main building comprises a machine shop and an inspection track bay. It is a substantial steel framed structure, with curtain-walls of brick. The machine shop is 316 ft. long by 40 ft. wide. The front 50 ft. is fitted for offices, with a mezzanine for light machine work overhead. The rear 60 ft. houses the blacksmith shop, a foundry, and a welding shop.

A 15-ton traveling crane (hand-operated from the floor) spans the shop and travels the full length. Alongside the machine shop is a lean-to housing a carpenter shop, a dip-room, and two ovens for the impregnating and baking of motor coils. The ovens are electrically heated and are provided with automatic temperature recorders and control. The inspection shed part of the building is 206 ft. long by 70 ft. wide and covers three tracks, each long

enough to take locomotive cabs, with ample working space between each. An inspection pit is provided the entire length of each track, with a cross-pit at each end for dropping wheels; these cross-pits are equipped with hydraulic jacks. A 30-ton electrically driven traveling crane with a 10-ton auxiliary hoist spans the main shed and travels the full length.

The shop is heated by steam, supplied from a separate boiler and power plant. Electric shop power is obtained from the traction lines and is converted in this plant by a small transformer and motor-generator set from single-phase alternating current to 250 volts direct current for the shop machines. The building also contains electrically driven air compressors and service and fire pumps. The water supply is from wells and is pumped into a 125 000-gal. steel tank mounted on a tower.

A modern equipment of individually driven machine tools is provided, including engine lathes of various sizes, milling machine, boring mill, slotter, shaper, planer, drills, wood-working machine, air hammer, and blacksmith tools.

The general storehouse is in a separate building adjacent to the machine shop, with a supply track in between. Alongside the main shed an outdoor locomotive inspection track with a pit is provided for locomotives not requiring shop service. The location of many of these various buildings and machines is indicated in Fig. 26.

Railway Company Telephone and Telegraph Protection

As already explained, the current in the trolley and feeder wires may induce currents in adjacent communication circuits. The principal effect of this is to raise the voltage of the communication line with respect to ground, with a possible break-down of its equipment, or electric or acoustic shock to operators. In addition, the communication lines must be protected against fire hazard due to possibility of high-tension and communication lines becoming crossed.

Telegraph Lines.—In telegraph lines, the earth is used for the return current path, and, consequently, an induced voltage in the telegraph wire, with respect to ground, may cause disturbance in the proper functioning of the relay and interfere with normal telegraph dispatching. To prevent this, therefore, in the local circuits, a low-resistance relay of special design was introduced. This has a split circuit so arranged that the 25-cycle current passes through both parts in equal amounts and hence does not operate the relay or sounder. To protect the operator against coming in contact with high induced voltages, this special relay is enclosed in a wooden box, and insulated knobs are brought through the cover. Furthermore, an insulated telegraph key is provided.

On the through circuits, resonant shunts are introduced around standard relays in order to reduce the voltage to ground when the telegraph key is open. This shunt by-passes the greater part of the 25-cycle current, so that the operation of the relay is not affected. Anti-resonant shunts, or 25-cycle chokes, are also installed in all circuits to limit the flow of current to ground.

There are no protective devices used outside the zone of electric train operation, and hence means had to be provided to confine the induction to

the electrified section of the road. This was accomplished by installing special choke coils and resonant drainage coils at each end of the electrified zone, which drain off the 25-cycle current to ground. This arrangement, while it prevents normal induction from going beyond the electrified zone, does not afford protection in case of short circuits; it is, therefore, supplemented by low-voltage lightning protectors, which are designed to drain off any excess voltage.

Telephone Lines.—In the ungrounded telephone circuits the induction is about equal in both wires, and hence little disturbance is caused by normal operation. However, to provide against break-down of protection, and possible shocks to users (at times of short circuit), and also against possible crosses of the open-wire telephone lines with high-tension power wires, insulating transformers are provided on the line side of all telephone instruments in the electrified zone. These protective measures have proved effective and sufficient for good service.

Design and Construction

It is obvious from the foregoing description that, in carrying out this extensive project, a co-ordination of design in several different branches of engineering—civil, mechanical, and electrical—is involved. Furthermore, as it is necessary that design and construction shall proceed simultaneously, a large organization is needed.

The Railway Company decided, having these facts in mind, to employ the writer's firm to carry out the project, this organization to become for the time being a special department of the road. Thus, the Company would receive the benefit of a staff of designers and constructors skilled in this kind of work, and the entire installation would become a first-hand railway company undertaking. The engineers reported to the Chief Engineer of the Railway Company as regards the general approval of plans and expenditures. Purchases and auditing of accounts were also handled through the regular railway departments in the usual manner. Thus, a close contact of interested officials with the work in progress was maintained without burdening them with a multitude of detail.

Designing.—A heavy electric system cannot be applied to an existing property without a considerable number of changes in the steam railroad facilities, and, generally, a modification of certain operating methods is required to produce best results. As a basis, therefore, for preliminary information, the Company's records and plats of alignment and grade were used, and many conferences with the officials were had in order to determine the electric train sheets, and the additional operating facilities desired.

The roadway cross-sections, the tracks, tunnels, and viaducts were carefully studied, as well as the buildings and other facilities, which might affect the type and clearances of the catenary structures. All were separately checked in much detail, and supplementary surveying work was required both at the start and as designing progressed. Notes were also made of the nature of the sub-surface materials as an aid in determining the depth and character of column and guy-anchor foundations for the catenary spans. As

the transmission lines were to be run through a mountainous and heavily wooded country, extensive surveys were required, and protracted negotiations for the purchase of property or easements had to be conducted.

In order to deal with all designing problems promptly, a field organization, under the direction of a resident engineer, was formed and directed from a centrally located headquarters on the line. This official had charge of all surveying parties, and later, when construction was started, he organized and carried out the work, reporting to the home office in New York.

Many special problems in producing the designs have been mentioned. The design of the power house in itself involved much detail, but the plans for the electrical distribution system for 137 miles of railway, required the largest number of designers. Practically each catenary span had to be dealt with as a separate problem, because of the continually differing local situations as to foundations, track curvature, number of tracks, clearances, etc., all of which produce varying stress conditions. In the aggregate more than 3 000 drawings were made for the entire installation.

An important consideration in preparing designs is to reduce the amount of field cutting and fittings of parts of the catenary system during the process of erection. This was accomplished by securing accurate preliminary information as to track alignment and elevations, and by careful laying out and calculation in the drafting-room, notably of the curve and overhead special work, such as the turn-outs and section breaks. Schedules and tabular information were made up and furnished the field forces for each catenary span, including cable sags and hanger lengths, prior to the distribution of materials on the job. In this way a surprising amount of time and expense was saved in the actual erection work.

Construction.—Space admits the mention of only a few of the methods followed, principally those involved in the production of the transmission and distribution. Aside from the main construction headquarters at the Narrows Power Plant, secondary headquarters were established at several points along the railway. These were generally in camp trains, which furnished mobile living accommodations for the men. As many as five of these trains were used at times in the different operations. Certain of them carried tower cars which were used in the erection of the catenary wires and fittings. Tent camps were provided, especially for the surveying and erection forces on the transmission line, where this work was remote from the railway.

The entire work was so organized as to require the purchase of a limited amount only of special equipment to supplement the Railway Company's work-train and other construction or service apparatus and tools. A locomotive crane with a 50-ft. boom was used in erecting catenary poles, setting concrete anchors, etc. Extensive grading, where required at the sub-stations, was let to local contractors who had the necessary equipment.

The tower cars were built in the Company's shops by erecting wooden frames with extension tops on standard flat cars. A number of special automobile gasoline motor cars were used to haul air compressors for field

drilling and for the use of the adjusting and tensioning gangs; also for inspection and the transport of men and light materials. These cars were light and could be quickly removed from the track, to accommodate traffic. The aim was always to interfere as little as might be with regular train operation, with due regard, however, to a fair utilization of the time of men engaged in construction operations.

Transmission Line Erection.—The tower foundations were grillages with foot sections set in earth excavations and back-filled ready for setting up the superstructure. All this part of the work was laid out so as to require the minimum amount of handling of heavy pieces over a difficult country. The erection of tower sections was by means of a gin-pole carrying a block and tackle. Horse teams were used for haulage and for hoisting operations.

The stranded wire cable for the transmission circuits was delivered on reels, each containing about 2 600 ft. and strung through snatch-blocks attached to the tower cross-arms. The cable was afterward sagged, as called for by the engineering schedule, using block and tackle. The determination of the proper sag was by sighting through targets on the towers and on the cables at mid-span.

The rate of progress in this work was about one tower erected per day. Transmission cable was strung and sagged at the rate of 6 miles of cable per day.

Catenary Erection.—In conducting this operation, experience is especially valuable; the work must be done "under traffic", much is special to its location, and the adjustments required are many and precise.

As heretofore mentioned, the pole foundations are of a character that is not only economical in material and labor, but eliminates the necessity of requiring concreting work trains on the line and the consequent interference with revenue trains. The foundation holes were dug, or blasted out, as the case might be, fitted with corrugated steel casings, back-filled, and capped, ready for quickly placing the poles. These were placed in position by a locomotive crane attached to the work-train which rapidly carried a number of poles. In this way frequently as many as 35 poles were erected in a 10-hour day. The guy anchors were delivered and set in the same manner. At the time of placing the poles in the foundations, they were wedged plumb by wooden blocks, which were subsequently removed when the casings were back-filled. After an anchor and pole was set it was unnecessary to use the crane to complete the erection of guys or fittings.

The various wires of the catenary system were strung in the following order, by use of extension-top tower cars in a work-train: First the balancing transmission wire, then the cross-spans, the messenger cable, and, lastly, the trolley and auxiliary wires. Proper tension was given, in accordance with the drawings, by the use of a dynamometer installed at varying distances, depending on the track alignment. An average of four reels of messenger cable, each containing 4 500 ft., was strung and sagged per day; this amount, however, varied considerably, depending on traffic conditions.

Next, a separate gang distributed and erected the hangers, which had been previously cut to the lengths required by the tabulations of the designers and had been equipped with top and bottom fittings. The degree of accuracy of the designing calculations was such that little field cutting was required. The hangers were applied to the messenger cable by an employee riding the messenger in a "boatswain's chair". Subsequent operations were as follows: The erection of insulator strings, the clipping-in of auxiliary and contact wires, and the final adjustments. All these were done from light tower cars which could be quickly removed from the track, when required.

At points where special work, such as over-head switches, section breaks, deflectors, and pull-offs, occurred the operations were more complicated, and progress was slow.

Apparatus and Materials Used

All equipment was selected with due consideration of the service requirements and suitability. The electric apparatus throughout was of Westinghouse Electric and Manufacturing Company design and make. The mechanical parts of the locomotives were furnished by the American Locomotive Company. The powdered fuel combustion system is known as the "Lopulco" and was furnished by the Combustion Engineering Corporation.

Operating Results and Economies

The Virginian Railway has now (January, 1928) had full electric operation between Mullens and Roanoke for more than a year and on the section between Mullens and Princeton for approximately two years. The actual performance under electric traction has fully met expectations and the results have been satisfactory from the operating standpoint. The locomotives and the power system throughout have met the severe requirements of service. Trains of 9 000-tons trailing load are regularly operated over the line at 28 miles per hour on the low-grade sections and 14 miles per hour on the heavier grades, and are held by regenerative braking on descending grades without the use of air-brakes. The time of run between Mullens and Roanoke (134 miles), with full tonnage, as compared with steam, has been reduced 7 hours per trip, as anticipated.

The effect of electrification in increasing the capacity of the railroad, especially on the heavy grade section between Elmore and Clarks Gap, has been so pronounced that it is not anticipated that the ultimate capacity of the single-track railway will be reached, even with continued increase in business, for a long time to come.

It would be of much interest to present figures for the relative cost of operation, steam and electric, and to deduce the savings effected by the changed form of motive power, but the writer believes definite figures must follow very careful analysis covering a considerable period of time, and the conclusions cannot be safely used as a generalization in favor of electric traction elsewhere; they must be understood to apply only to the character of conditions, physical and operating, of the road in question.

It can now be safely stated that, as concerns the Virginian Railway Company's installation, savings have been very large and establish the fact that they will suffice to pay a very satisfactory return on the cost of the project, aside from a consideration of the indirect advantages secured. This result is largely due to the peculiar conditions of the service, that is, very heavy trains on a long division, operated at relatively high speed, such that the time of enginemen and train crews is greatly reduced over what is possible in a congested and slow steam operation.

Speaking in very general terms only, the main savings in operating costs on the electrified division of this road comprised reductions in three very large items, so that, on the basis of equal gross tonnage moved, the relative cost of electric operation and steam operation is as follows:

Enginemen and trainmen	One-third that of steam.
Locomotive repairs	One-third that of steam.
Fuel	One-half that of steam.

Engine-house expense, which is a comparatively small item, was reduced to one-quarter the amount required by steam service.

Personnel

This electrification was under the general direction of H. Fernstrom, M. Am. Soc. C. E., Chief Engineer of the Virginian Railway (now retired); Hugh Pattison, M. Am. Soc. C. E., Engineer of Electric Traction; John W. Sasser, Superintendent of Motive Power; and J. E. Sharpley, Electrical Engineer. The engineering, detail designing, and the construction work was carried out by Gibbs and Hill, Consulting Engineers, New York, in whose organization, in addition to the writer and E. R. Hill, M. Am. Soc. C. E., directly in charge of the designing and construction work, were Messrs. S. A. Spalding, Chief Engineer; Norman Litchfield, Mechanical Engineer; H. S. Richmond, Structural Engineer, and H. W. Hudson, M. Am. Soc. C. E., Resident Engineer, in the field.

DISCUSSION

W. B. POTTER,* M. AM. SOC. C. E. (by letter).—A substantial increase in tonnage capacity of the Virginian Railway with a material reduction in the number of locomotives and operating expenses is a result which speaks for itself. A comparison of the capital charges and operation under steam and electrification will be awaited with interest. A further comparative estimate based on a steam service equivalent to the heavier electric service might be even more interesting.

The electric system chosen with its constant speed characteristic appears to be well adapted to the particular requirements of this division of the Virginian Railway. It does not follow, however, that this is the only electric system capable of meeting the requirements.

Regeneration, as in this case, would seem to be a very desirable feature for any heavy grade operation. The return of energy thus made possible is in the interest of economy. However, is not the elimination of air-braking, with its consequent wear and tear on the rolling stock, an equal, if not greater reason for using regeneration? By attention to dispatching, the regenerated energy might be expected to reduce the fluctuation in power demand, but, after all, the purpose of a railroad is transportation and in an issue between power demand and train movement, it is likely that the trains would have the right of way.

Aside from conditions to the contrary, it would seem as if a power company having a diversity factor, and, therefore, a better load factor, should be able to sell power with profit for at least the same price as the cost of producing it in a station having only the railway load. The fixed charges, in particular, would be reduced by the better load factor.

The results obtained from pulverized fuel with the widely fluctuating demand for steam, appear fully to justify its use. It is an excellent showing for pulverized fuel under these extreme conditions. The diagram (Fig. 7), showing how uniformly the steam pressure is maintained under the operating requirement, is certainly an excellent record.

Successful current collection may be defined as the absence of sparking between the collector and the contact wire. With a system of overhead suspend wire having the essential "contact quality", it has been fairly demonstrated that several thousand amperes may be taken from a collector without visible sparking. In fact, current collection with more than four times the amperes per collector on the Virginian, has been in successful operation for a number of years. It is widely recognized that sparking between the contact wire and collector has a far more destructive effect than the wear resulting from mechanical friction.

The use of side-rod drive for electric locomotives would seem to be more of a special application, and although well suited to these particular locomotives, it seems probable that the trend of locomotive development will be more toward the use of rotary parts only. The requirements as to the strength of rods and side-frames in the motor-drive locomotive are considerably more severe than

* Chf. Engr., Railway Eng. Dept., General Electric Co., Schenectady, N. Y.

for a steam locomotive of corresponding tractive effort. That these electric locomotives have been so successful is to the credit of both those who designed them and those who built them.

The use of locomotives weighing 78 000 lb. per driving axle so far exceeds the general practice that the effect on the track is worthy of careful study. It may possibly be found that this weight, in an electric locomotive with a true running balance, will be comparable in its action on the track with a steam locomotive having less weight, but with an unbalanced rotating factor.

Every railroad electrification has successfully performed the service for which it was intended, and the Virginian is to be congratulated as a successful demonstration of the heaviest service yet undertaken.

MARCEL JAPIOT,* ESQ. (by letter).—Although the writer had the pleasure of investigating in detail several years ago the wonderful electrifications of the Pennsylvania Railroad and of the Norfolk and Western Railway, he did not find the opportunity to inspect the Virginian Railway electrification. Nevertheless, it may be of interest to know how the American situation in the matter of main-line electrification may be viewed from the other side of the water.

The most striking result of the Virginian Railway electrification is certainly the increased capacity of the line. It nearly doubled the speed of the heaviest trains in the world, on such severe grades as those prevailing in the Alleghany Mountains. That is enough to show that the decision to electrify was a most wise one, even if the direct savings had not paid a substantial return on the investment.

The writer agrees fully that, considering the magnitude of the problem, the single-phase overhead line was the best solution, and even perhaps the only one available at the present stage of development. That conclusion may seem strange, coming from a French engineer who advocates direct current for main-line electrification in his own country; nevertheless, it is very easy to justify it.

No system of electrification may be said to be the best in itself; each problem must be examined separately, and the engineer must look, in each case, for the system best suited to the special conditions involved in that individual problem. That was exactly the position that French engineers took when they selected the direct current at 1 500 volts as standard in France.

However, with such extremely heavy loads as those to be cared for in the Virginian Railway electrification, there is no doubt that it was necessary to raise the voltage on the contact line as much as possible, in order to bring down the amperes to be collected by the electric locomotives. Therefore, the single-phase overhead line, with its much higher voltage, seemed to be a necessity in that case. The Virginian Railway, with its heavy trains and its severe grades, was indeed the most appropriate field to be found anywhere for the single-phase system.

If the conditions of the problem are changed, the decision becomes reversed. Such was the case on the Illinois Central Railroad, where the scale turned, as it did in France, in favor of the direct current at 1 500 volts.

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While the electrified lines in the United States are more important than anywhere in the world, they include only a small part of its vast system of railways. Moreover, there have been, until now, very few physical connections between electrified trunk lines using different types of current, alternating and direct. These two facts seem to explain why American engineers did not feel any urgent need for standardizing on a definite type of current for main-line electrification, as did several European countries.

With the unrestricted liberty still prevailing in the American field, engineers in the United States have the opportunity to give to each individual problem the most favorable solution. On the other hand, standardization must always be a compromise between the advantages and the drawbacks of the different systems of electrification for several various problems.

The necessity of such a compromise does not seem to be imminent in the vast territories of the United States. Nevertheless, it appears from the recent developments that, if any compromise had to be made at an early date (which does not seem to be probable), the single-phase overhead line might be adopted successfully in America.

That does not mean that the writer is ready to depreciate in any way the advantages of the direct current for many definite problems of electrification; but nobody can deny that the much higher voltage now allowed on the contact line by the single-phase alternating current is of the utmost importance for the average conditions prevailing on American railways.

Moreover, the flexibility of that system for electric motive power must be kept in mind, because it permits the use of either (1) the straight single-phase system, as on the Pennsylvania Railroad and the New York, New Haven, and Hartford; (2) the so-called "split-phase" system (single-phase to three-phase), as on the Norfolk and Western Railway and on the Virginian Railway; or (3), the motor-generator engine (single-phase to direct-current), as on the Detroit, Toledo, and Ironton Railroad, and on the Great Northern Railway. The motor-generator type may perhaps be further improved in the future by using some kind of static or rotating rectifier, instead of the somewhat cumbersome motor-generator set. In any case, the diversity of the solutions available for electric motive power with the same single-phase contact line gives to engineers great facilities to adapt that system to very different conditions, which is a most important quality for a compromise.

Nevertheless, from an engineering point of view, it is fortunate that the necessity for such a compromise has not yet become urgent in the United States, as it gives to American engineers an unlimited field of possibilities for other wonderful achievements.

L. K. Sillcox,* M. Am. Soc. C. E. (by letter).—One of the greatest disadvantages of the past for those endeavoring to present a proposition favoring electrification of an existing steam-operated railroad has been that their proposals were based on theory which made it possible of attack by contending that the theory would not be borne out in practice. Mr. Gibbs has been able

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to avoid such a situation by reason of his long, valuable, and successful experience in connection with the electrification projects carried out on the Pennsylvania, Long Island, and Norfolk and Western Systems, as well as a number of other well known and important undertakings. His contribution to the art of electric railroading has always been worth while, and this paper covers the case accurately and fully.

About 1926, the writer was privileged to make a survey of this line and, therefore, can present some first-hand impressions gained at that time. In the first place this railway has a capacity for dumping 11 400 tons of coal per hour at the seaboard, which is equal to 273 600 tons per 24-hour day. It has always been recognized as a leader in the movement of heavy-tonnage trains, and in the operation of most powerful modern motive power. In May, 1921, all records of heavy trains handled were exceeded when there was a test run on the Third District, consisting of a train of 100 loaded cars, each with a capacity of 120 tons, aggregating 16 000 gross tons. To give some idea of the rapid expansion of the road's business, it is interesting to observe that, in 1911, a coal tonnage of 2 141 009 was handled; in 1920, this figure was increased to 7 621 555 tons; in 1925, it went to 8 250 000 tons; while in 1926 the total was 11 500 000 tons.

It is well, at the outset, to take seriously the author's remark that "it is obvious, however, that uniformity [in energy] demand cannot be secured, even approximately, in practical railroading", having due regard to the factors producing reasonable plant cost and economy of operation. When there is sufficient tonnage available, one of the principal features which limits the earning capacity of any railway system is the total amount of tonnage that can be handled over the line in a given time. The project under discussion was undertaken only after most careful consideration of all the comparable methods of successfully handling and expanding competitive traffic demand, and its greatest significance is an economic one.

The electrification of the Virginian Railway has, in general, enabled a much better use to be made of the capital expended in the construction and development of the property up to that time than would have been attainable under any other means involving an equal expenditure. This is mainly due to the more prompt movement of tonnage and greater utilization of the property as a unit than formerly was possible. Under electrified traction the section from the Elmore, W. Va., Yard to Roanoke, Va., a distance of 132 miles, is operated as a one-locomotive district. Under steam operation this was a two-engine district with turn-around runs between Princeton, W. Va., and Elmore. Trains of 6 000 tons are being handled from Elmore to Clarks Gap, W. Va., with one leading locomotive and one pusher behind, each composed of three electrical units. Under steam operation, trains of 5 500 tons were hauled by three Mallet engines, one in front, of the 2-8-8-2 type with a tractive effort of 101 300 lb., working compound, and two pusher engines of the 2-10-10-2 type with a tractive effort of 147 200 lb., working compound. The speed under electric operation is 14 miles per hour, while under steam it did not exceed 7 miles per hour. At Clarks Gap the load of the trains was increased to 9 000

tons, which was taken to Princeton with one locomotive. From Princeton to Roanoke, the train was assisted by a helper, from Whitethorne to Merrimac, Va., and also out of the Princeton Yard. Under electric operation this helper service is eliminated. The speed of the trains varies between 14 and 28 miles per hour, with a consequence that this line, which is of single-track construction (except from Mullens, W. Va., to Clarks Gap), is equipped for a handling capacity of 12 500 000 tons per year, and when this figure is reached the tonnage is to be carried by forty-eight electric units, or sixteen 3-unit locomotives. This same service, if it were handled by the existing design of steam power all of which was of modern construction, would require approximately twenty of the large 2-10-10-2 type Mallets, and forty-four of the 2-8-8-2 type Mallets, a total of sixty-four steam units as against sixteen electrical units; and, in addition, under steam operation, extra helper engines to assist trains out of the Princeton Yard and between Whitethorne and Merrimac would be required. The regenerative system of braking on grades, especially in the case of tonnage trains, is also of great value from a practical railroad-operating standpoint.

The electric locomotives have a 2-8-2 wheel arrangement, with a weight per unit of 212½ tons, so that the weight of the 3-unit road locomotive is 637½ tons. The maximum tractive effort of each unit, as limited by electrical capacity, is 92 500 lb., or 277 500 lb. for the 3-unit locomotive, which occurs at an adhesion of slightly less than 30 per cent. Computing the conservative starting adhesion at 25%, the tractive effort is approximately 77 000 lb., or 231 000 lb. for the 3-unit locomotive. This represents a starting force 57% greater than the compound rating of the 2-10-10-2 type Mallet steam locomotive (one of the largest steam engines ever built) and 31% greater than this engine when working simple. The continuous rating of the 3-unit locomotive is 135 000 lb. at 14 miles per hour and 78 800 lb. at 28 miles per hour. In the high-speed connection of the motors the three units exert 6 000 h.p. continuously. These engines operate on 2% grades and through a maximum line curvature of 12 degrees.

One feature of the project which is of unusual interest and needs to be especially commended has regard to the satisfactory location and thoughtful study given the design and equipment of the shop facilities at Mullens, which will enable the motive power to be kept in proper condition at a minimum of expense and to give a maximum of reliable movement. The greatest need under electrification, as compared to steam operation, is that of comprehensive inspection, since that requires much the greater part of the time of running maintenance forces; whereas with steam motive power, defects are relatively self-evident upon arrival at the terminal, or during the running inspection, and it is the time to make repairs which predominates and not that of inspection. For this reason, it is most unwise for any steam railroad, which may contemplate electrification, to attempt any lack of due consideration, required to maintain electric motive power efficiently, because the two types of operation do not lend themselves to similar treatment in this respect, and the facilities must be set up with a view to the peculiarities of the requirements

in each case. The ideal of attainment in transportation is to have all trains run at the same speed, and although that ideal is not fully realizable, it can be more nearly attained with electric than with steam traction.

I. ÖFVERHOLM,* ESQ. (by letter).—One of the reasons given for the 88 000-volt transmission line and the seven sub-stations is to make it possible to avoid the stub-end feed and its bad effects on the communication circuits along the railroad. As far as the writer is informed, the result of these arrangements is not fully satisfactory. It seems that, in any case, it has been necessary to make the telephone and telegraph circuits immune from the residual disturbances in these wires.

The writer, therefore, thinks that it would have been better to provide some other means in order to get a more desirable location for the communication circuits. According to Swedish practice, the transmission lines and the sub-stations would not have been designed for 88 000 volts. Swedish engineers would have used a trolley line with a potential of 16 000 volts and a balancing wire such as that used on the Virginian Railway, with a potential of 32 000 volts. Thus, they would have had a transmission potential of 48 000 volts, which seems to be great enough for the transmission along the electrified part of the Virginian Railroad.

The trolley wire and its catenary, according to current practice in Sweden, would have been made of copper with a section of 300 000 cir. mils for both together. The balancing wire would have been made also of copper and with the same section, and then a feeder line for 48 000 volts with two wires each with a section of 300 000 cir. mils would probably have been used. This causes a very severe stub-end feed, which is not to be feared with the precautions now used.

A fair comparison may be made with the most recently electrified line from Stockholm to Göteborg, where very satisfactory experiences were obtained with similar arrangements. On this line, the communication cable belonging to the railroad (which cable is placed in the roadbed), has a disturbance potential, under normal service, running up to a maximum of 10 to 15 volts, and at most severe short-circuits, 47 volts. For this reason there are no disturbances in telephone and telegraph circuits even at short-circuit. The telegraph circuits can be, and are, operated with a ground return on a single wire in the cable.

For such a case as the Virginian, Swedish engineers would have used feeder booster transformers inserted in the trolley wires and the balancing wire midway between the balancing transformers, which would have been spaced at distances of about 6 miles. In this way the arrangement of the transmission would have been much simplified, and the cost would have been reduced. Also, the residual disturbance voltages on the communication wires would not have amounted to such values as are reached on the Virginian Railway. However, these arrangements might give a somewhat greater potential drop in the contact lines.

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RALPH BUDD,* M. AM. SOC. C. E. (by letter).—The Virginian Railway is a good example of traffic and operating conditions which justify electrification purely for reasons of economy. The expedited movement of freight, the saving in fuel and in wages of trainmen and enginemen, and the avoidance of expensive additional tracks for increasing the capacity of the railway combine to make current economies which represent a large return on the investment. Undoubtedly, there are more instances in the United States where electrification of heavy traffic sections of steam railways is justifiable for similar reasons, and like action, therefore, may be expected to follow. On most of the railway mileage of the country, however, the traffic density is not great enough to pay a return on the cost of electrification sufficient to make the investment attractive.

The improved efficiency of the steam locomotive makes it more difficult to show great economy in electric operation, as compared with steam, than it was in 1918 and thereabouts. Of course, any study to be convincing to a competent and careful railway board of directors, must compare the estimated cost of operation after electrification with the cost of operation using the most modern and best adapted steam locomotives for the duty imposed, rather than the steam locomotive performance some years ago, or even that of the present. This is a point of some importance because, pending the final solution of a transportation problem, even the best railway company may wisely hesitate about replacing its old steam locomotives with strictly modern ones.

As Mr. Gibbs has stated, the Norfolk and Western electrification is so closely related to the Virginian, both in location and in operating requirements, that the very successful record of the former naturally influenced the adoption of the same system on the Virginian, thus giving convenience in detouring or the exchange of motive power, if desirable.

Whether or not rod-type electric locomotives are the most desirable does not seem to be demonstrated by anything in this electrification, nor indeed is there anything to show whether locomotive performance, and especially locomotive upkeep, is more favorable with this particular system of electrification than it would be with others. The limited speed control does not seem to be any particular handicap here as it might be under other and more varied conditions. The most striking differences the writer has seen in comparative data on heavy-duty electric railway operation have been in the cost of maintaining the motors, the best showing being made by direct-current motors. That item, of course, is only one, although a very important one, of many which go to make the total of operating cost. In any event, the Virginian Railway undoubtedly has a highly efficient and economical electrification.

GUIDO SEMENZA,† ESQ. (by letter).—The author is correct when he states that the electrified section of the Virginian Railway furnishes an example of the heaviest electric traction development to be found anywhere in the world. This is true for the amount of traffic hauled, for the magnitude of the locomotives, and for the power required by each unit. These features are of such

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† Engr., Milan, Italy.

importance as to make this paper one of greatest interest, and the results obtained, both technically as well as economically, are so satisfactory that Mr. Gibbs and his collaborators are to be complimented.

In this clear exposition the author shows that the problems to be solved were quite unique—a very difficult profile of the line, with heavy grades; a traffic of such a character as to be called a "shuttle service"; a small number of very heavy trains, running at a relatively low speed; and a uni-directional movement of the charge. Therefore, if the methods, systems, and devices used in solving the problems of this large project appear to be justified and appropriate, one must be careful in extending them to other cases, in which the conditions of traffic are quite different.

The power of 11 000 kw. required by each train justifies the alternating high tension in the contact wire and, although at first it might seem a useless complication to transmit single-phase current when the motors are three-phase, one can only approve of having done away with the serious construction complications and the severe and costly operation of a 10 000-volt, three-phase contact line.

To one who has had occasion to follow closely a rather extensive system of three-phase traction, it seems rather queer that the three-phase motor was chosen in order to meet the severe starting conditions. The three-phase motor has many good qualities, but its drawback has always been considered to be in its starting characteristics, namely, a limited torque, that cannot be improved by slowing down, and that is highly influenced by the line voltage drop. It is only when the starting is difficult and when the track conditions are bad, that an abnormally high torque is required, with a corresponding increase of current, and the voltage drop then is maximum. Moreover, the starting conditions are likely to be altered by the heating of the water rheostats.

If, in spite of these structural features of the three-phase motor (and it does not appear that the phase converter can improve matters very much), the operation is satisfactory, that means that the dimensions of the motors and rheostats have been calculated so as to meet these special conditions. It would be interesting, however, to compare (as to weights, costs, and efficiencies) the actual use of three-phase to that of direct-current motors in these same locomotives.

F. H. SHEPARD,* ESQ. (by letter).—The paper shows, in a comprehensive way and in valuable detail, this important and successful railway electrification.

The institution of this undertaking reached its final stages in November, 1922, with the request from the Railroad Company for a tender upon and a recommended plan and method for electrification. This was required in the form of (1) a competitive bid for the apparatus and equipment involved (as manufactured by the Westinghouse Electric and Manufacturing Company); (2) an estimate on the other items going to capital expenditure; and (3) an estimate of cost of operation and savings. The contract was executed under date of April 18, 1923.

* Director of Heavy Traction, Westinghouse Elec. & Mfg. Co., New York, N. Y.

Subsequent to the signing of the contract, Mr. Gibbs and E. R. Hill, M. Am. Soc. C. E., took over the execution of the field work, the design and construction for the power house, sub-station, transmission, and conducting systems, all to suit the requirements for traffic movement as specified in the plan of operation.

A most gratifying part of the whole undertaking has been the confirmation of operating results and advantages which were predicted for this electric operation, all of which were realized with ample margin. The engineering and construction work undertaken by the consulting engineers was planned and executed with smoothness and despatch and in co-ordination with the regular operation of the road under steam.

From the initial run of the first train until the whole section was under complete electrical operation, the addition of electrical units was attended with a distinct and regular improvement in the movement of traffic. The members of the Operating Staff of the Railway Company were alert and responsive in their desire to learn the details of electric operation and had so far mastered the handling of the new motive power that little or no assignment, apart from the road's regular forces, was necessary for instruction of the forces required to operate and maintain the electric locomotives in this unprecedented train service.

As this operation now stands there has been realized an increase in capacity for moving traffic materially in excess of that claimed for the undertaking. Furthermore, the locomotives have capacity for service materially in excess of that required. They can make the run between Clarks Gap and Roanoke in approximately 1½ hours less time than that stipulated; they can handle the specified tonnage up Clarks Gap Hill even if the grade were 5 miles longer; they have demonstrated a momentary capacity for initial start equivalent to moving 6 000 tons on a 2% grade with a single locomotive; and, in addition, they have demonstrated the entire success of close regulation of tractive effort for these exceedingly heavy trains under all conditions of starting and running.

E. R. Hill,* M. Am. Soc. C. E.—The Virginian Railway, built in 1905-07, was planned and constructed in a far-sighted and broad-gauged manner, as a result of which grades were made as favorable as possible considering the region traversed. There were no unnecessary dips or other objectionable features, and only two pusher grades under steam operation and one under electric operation. The bridges were constructed for heavy engine and car loads and thus even before electrification the heaviest trains in the country, and, in fact, in the world, were operated on this line. In view of this and the physical limitations in strength of draft-geared cars in electrifying, it was not thought advisable at the outset to adopt maximum train weights (6 000 tons on Clarks Gap Hill and 9 000 tons elsewhere) materially greater than had been handled under steam operation, although these weights are about 50% greater than could be handled by steam in winter. It will be noted that provisions have been made to increase the trains to 12 000 tons at such time in the future as

* Cons. Engr. (Gibbs & Hill), New York, N. Y.

traffic conditions, construction of cars, and other considerations make this desirable or practicable.

The paper refers to the exacting conditions of the load and the probable difficulty of carrying such a load on a general power supply system. From the nature of the load it would be difficult to carry it on any ordinary power system except a very large one, a much larger system with a heavier power background than was available in that locality at the time.

The choice of the split-phase type of locomotive over the motor-generator type or the straight single-phase was somewhat influenced by the facts that there were two railroads in that same valley; that certain combinations of railroads were and are now permissible; and that mutual interests pointed to a uniformity of system and locomotive characteristics as being desirable in that territory. In fact, during the World War, these two roads were operated as one railroad. There were other considerations, of course, which led to the selection of the split-phase type of locomotive which it is believed was the right one under all the circumstances. Furthermore, at that time the motor-generator type of locomotive had not been developed to the point where it could safely be adopted on a large scale.

With regard to the sub-stations, it should be noted that all the power which goes to the trains passes through the main transformers, the balancing transformers being part of the distribution system and affecting the regulation. With this arrangement of transformers, less main transformer capacity is required than with a more frequent spacing of ordinary transformers, partly because of the necessity for having a "spare" at each location. The system was laid out in both ways, and the most economical and efficient one, all things considered, was selected.

The electrification of the line cost approximately $15 000 000,—the amount estimated. Allowing credit for the additional facilities that would be required to secure approximately the same capacity by steam, including double-tracking, locomotives, and other facilities, the net additional cost of electrification was about $5 000 000. The purpose of this electrification was to secure increased capacity of the railway, reduce operating costs, avoid expensive and difficult changes in the way of double-tracking, secure smoother, safer, and more reliable movement of trains, and, incidentally, to clarify the air in tunnels, especially the Merrimac Tunnel at the summit of the Alleghany Mountains, which is about a mile in length.

After 1½ years of electric operation it can be said that (1928) the expectations have been more than realized in all respects. The direct savings in operating costs effected by the change, as determined from carefully compiled steam-operating costs for this particular division prior to electrification and from actual results since, indicate a decrease in such items of cost per gross ton-mile of more than 50 per cent. This saving on the gross ton-mileage operated last year (1927) was sufficient to pay fixed charges on the gross cost of electrification, or three times that required to pay such charges on the net additional cost of electrification. The operating and the financial results, therefore, have been entirely satisfactory.

NORMAN LITCHFIELD,* ESQ.—The speaker has been closely associated with the Virginian Railway electrification throughout, and the following remarks are more in the nature of emphasis than discussion.

One of the outstanding features of the electrification is the rugged character of the country. Those who are unfamiliar with West Virginia can have little conception of the topography, for while the mountains are not of the height and grandeur of other great chains, nevertheless there are a great many of them, placed in such irregular groupings that it almost seems as if Nature had had a great convulsion there. This made the running of the transmission line a difficult matter, particularly in the delivery of materials to the site, new roads having to be cut in many cases.

The climate is more severe than is ordinarily associated in one's mind with railroads in that latitude, making winter work, although generally possible, at times, quite difficult. The "frozen fog" as its name implies is neither ice nor snow, but a sort of frost piled up on the wires, which sometimes attains the diameter of a man's arm. When the sun strikes such a coated wire in one spot, the frost drops off rapidly in the sunshine, but remains on the wire in the shade. This results in an unbalanced condition and the wire whips upward. To take care of this the lower wires are offset from the upper ones, as is shown in Fig. 12.

The diversified character of the problems encountered in a major electrification is apparent from Mr. Gibbs' paper. Virtually every branch of civil, electrical, and mechanical engineering is involved. One of the most striking features was the necessity for anchoring the power-house foundations to the solid rock to prevent possible dislodgment during river floods. The very patent question might be asked, "Why put the power house in the bed of a river?" The answer is obvious in view of the nature of the surrounding country, that this was the only flat place available, where condensing water could be had.

Some idea of the magnitude of the transportation problem which the Virginian Railway has to meet can be given by a more detailed consideration of the size of the train. Unless one is a railroader it is hard to conceive just what a 6 000 or 9 000-ton train is. As the cars average about 46 ft. in length and weigh (including contents) at an average about 100 tons, a 6 000-ton train contains 60 cars or more and is about 2 800 ft. long. Similarly, a 9 000-ton train has 90 cars and is about 4 100 ft., or nearly ¾ mile, long. If this were composed of the cars one finds in an ordinary freight train, averaging, say, 50 tons each, it would include about 150 cars and be about 6 000 ft. long. This gives some conception of what a Virginian Railway train really is.

To start a 6 000-ton train on the "hill" may require a tractive force of more than 500 000 lb. with one locomotive at the front of the train and another at the rear, pushing. At present, there are no means of communication between the front and the rear locomotive other than by whistle signals, which often cannot be heard because of intervening hills; hence, it is virtually impossible to start both locomotives at the same time without recourse to an indirect practice. The brakes are applied on the pusher locomotive, and the front

* Mech. Engr., Gibbs & Hill, Cons. Engrs., New York, N. Y.

locomotive backs the train down into the rear one. A train of this length will have a total slack in the couplings and draft gear of perhaps 50 ft., or more. Hence, the rear engine receives a relatively heavy bump when the slack has been taken up. This is the signal for the rear engineman to apply his power. Even then it is difficult to synchronize with the front locomotive, and some little time may elapse before the rear engine starts moving, in some cases as much as 1 min. This, of course, puts an exceedingly heavy electrical strain on the motor, which is one of the features that makes the induction motor a proper selection for this class of service.

Equally interesting is the action of the locomotive in descending the long 1.5% grade. An ordinary freight train hauled by a steam locomotive, going down grade sways back and forth both sideways and "fore and aft". This is largely due to the fact that it is impossible to keep the brakes applied continuously, because this would burn up the brake-shoes, crack the wheels, and cause havoc generally. With the exception of a few cars on which retainers are set up, it is necessary to release the brakes after the train has slowed down to as low a speed as possible without danger of coming to a full stop, then release the brakes again and let the train gain headway until the maximum safe operating speed is reached. This process is repeated again and again. Although the air-brake has been improved so that it is a marvelous mechanism, it is not perfect, and it takes a relatively long time for its action to be transmitted from the front to the rear of a long train; also, irregular braking action occurs and, in such a case, it is not unusual to break the train in two.

With the electric locomotive, after the train gets about on the crest of the hill, the engineman simply throws a lever, and the motors begin to regenerate, thus acting as a powerful and steady brake, so that the train proceeds down the hill at a practically constant speed, and with little irregular motion. This, of course, gives a very desirable operation, and one which is much easier on the rolling stock.

The electrification is popular with the rank and file of the railroad men (as shown by the expressed desire of the enginemen to obtain the electric runs) and has generally proved its worth.

SIDNEY WITHINGTON,* ESQ.—In presenting as full information as is possible within a brief space, of the large and important undertaking represented in the Virginian Railway electrification, Mr. Gibbs' paper is of great value as a matter of record.

One of the outstanding features of the Virginian electrification is the extreme variation of load over a short period of time, in swings amounting to nearly 50 000 kw., as indicated on the load curve in Fig. 4. Only very large commercial systems would be able to handle such great and extreme variations of load, and the wisdom of choosing pulverized fuel to obtain flexibility at the power plant is obvious. In a plant of this size, with such load characteristics, the consumption of 1¾ lb. of coal per kw-hr. net output is a creditable performance.

* Elec. Engr., N. Y., N. H. & H. R. R., New Haven, Conn.

In connection with the regeneration of power on descending grades, it is of interest to note that the only instances in the United States where such regeneration takes place occur in territories where, on account of the proximity to coal fields or availability of hydro-electric power, the unit cost of energy is relatively low and the money value of the regenerated power is of little importance. The advantage of regenerative facilities is not so much in the power saved as in the increased safety of operation and ease of handling trains. It is noteworthy that, in many instances abroad, regeneration as such has been discarded and replaced by simple electric braking wherein the traction motors operate as generators as in the case with regeneration, but the energy is absorbed in resistance on the locomotive itself and is thus not utilized.

Very much the same idea of balancing transformers was applied in 1913 on the New York, New Haven and Hartford Railroad between New Haven, Conn., and New York, and on the New York, Westchester and Boston Railway, a subsidiary. This type of transformer on the New Haven combines the benefits of 22 000-volt transmission with the balancing effect to reduce interference with telephone circuits. The plan has also been used abroad to some extent.

Poles of H-section were utilized on the New Haven, in 1914, in side-track construction, and, in 1925, on the main-line track between South Norwalk and Danbury, Conn. This type of pole, as Mr. Gibbs suggests, has great advantage in the use of the flanges in climbing, as the climbers or "skates" devised for the linemen eliminate the necessity for ladders or steps as in the case of tubular poles.

In connection with the electrification of the New Haven Line between South Norwalk and Danbury, a note on the foundation design for the catenary columns may be of interest. The poles or columns were the H-type, of normal sizes, 6, 8, 10 and 12 in., depending on the loads imposed. The foundations were poured in place, and embedded in the concrete of each pier were two channels, set back-to-back and separated by a distance equivalent to the dimension between flange faces of the pole to be set. These channels, and the lower end of the pole, carried holes for 1-in. bolts in sufficient number to develop the required strength of the connection. In the process of setting, the poles were slipped down between the channels and enough bolts installed to hold them, before the work-train with the crane moved on to the next pole. In spite of the fact that a considerable amount of both freight and passenger traffic was operated in both directions on this single-track line (which it was necessary to clear sufficiently in advance in order to avoid delays), as many as forty-nine poles were set in a single day in this manner.

Fig. 19 shows the limitation of the inclined type of hanger installed on extremely sharp curves. By laying a straight-edge along the contact wires between pull-off points shown in the photograph, it may be noted that the contact wire assumes nearly the same position which would be the case in chord construction mentioned by Mr. Gibbs, and it has been the experience on the New Haven that for curves sharper than 4°, the chord construction is as advantageous as the inclined hanger type in the matter of length of spans, is somewhat easier to install, and more economical to maintain.

In connection with the design of the catenary system, it is necessary to exercise care to insure that the cost of carefully developed details in the drafting-room, as suggested by Mr. Gibbs, covering all possible contingencies in the field for each catenary span, is not in excess of that which would be saved by a small amount of additional work done by the field gangs which might be necessary with standardization. It is probable even that some standardization of span details would be justified in actually saving money in the field both in construction and maintenance, by eliminating large numbers of odd lengths of hangers and consequent heavy storehouse expenses in handling material.

Under electric operation the engine-house expenses are noted as being about one-fourth of those accruing under steam operation. This seems a somewhat high ratio unless the Virginian Railway charges more to the account of engine-house expenses than some other railroads. The ratio is usually nearer 1 to 6, or even 1 to 10, on account of the simplicity of electric locomotive terminals. They eliminate the need of coal and ash-handling facilities, and the savings on this account may amount to a considerable sum in the aggregate.

GEORGE J. RAY,* M. AM. SOC. C. E.—The speaker wishes to add a comment as to the regenerating features of operating electric locomotives on heavy grades. The handling of heavy freight trains on steep descending grades is a difficult operation. On the road with which the speaker is connected there is one stretch of 17 miles of 1½% grade. On this grade it is necessary to restrict the speed of heavy freight trains to 15 miles per hour in order to avoid "runaways". The customary practice makes it necessary for the engineman to make approximately thirty-five brake applications and releases during the descent; that is, he must apply the brakes, bringing the speed of the train down to about 10 miles per hour, then release them again, and allow the train to run free until it reaches a speed of about 18 miles per hour and then apply them again. With this in mind, the great advantage in the regenerating feature under electrical operation is readily understood.

There has been much discussion about the feasibility of operating trains under automatic train control on such heavy grades. When the brakes are applied at low speed it is necessary to make a full stop before releasing them; otherwise, there might be a catastrophe. On a train of 85 or 100 cars it takes so long for the air to act from the engine to the rear of the train that if an engineer is permitted to release at slow speeds, especially with empties, he is almost certain to buckle the train or pull it in two. It is clear, therefore, that the introduction of an automatic brake application might be a rather serious matter where heavy trains are being handled down a steep grade. There would seem to be less likelihood of trouble with electric traction where the train is operated at a uniform rate of speed such as that indicated by the author. The speaker would like to inquire whether the question of automatic train control was given consideration in connection with either the Virginian or the Norfolk and Western electrification.

* Chf. Engr., D. L. & W. R. R., Hoboken, N. J.

JOHN P. HOGAN,* M. AM. SOC. C. E.—The speaker feels that if there were more knowledge of the advantages of electric operation among civil engineers there would be more electrification. Mr. Gibbs has gone a long way toward making this knowledge available and all civil engineers owe him a great debt of gratitude.

The Virginian Railway has a very heavy traffic; in fact, the maximum of 110 000 tons total weight, in both directions, over a single-track line in a single day, is very close to the maximum in the United States, and, therefore, in the world. Outside this Continent a 2 000-ton train would be considered a curiosity, yet here is a system planned for 9 000-ton trains and the speaker understands that even heavier trains have recently been hauled.

Mr. Gibbs has touched generally on the conditions which led to the construction of an independent power plant. While it is possibly true that, in the district in which the Virginian Railway operates, the existing utilities could not handle a load of this character, it may be stated in general that, in the more settled parts of the country, it should always be more economical to combine the railway load and that of the utilities. Whether it is possible to do so is always dependent upon the arrangements that can be made and whether a discussion of the relative costs can be brought about on a basis of perfect frankness and honesty. There is a great deal of misunderstanding between the power companies and the railroad companies on the question of power supply which is about equally distributed between the two and is generally due to misinformation.

The power companies do not often take the trouble to acquaint themselves with the load characteristics, nor do they do the necessary work to determine the effect of this load on their system, and the result is that uncertainty naturally causes high quotations.

The railroad people, on the other hand, frequently impose restrictions on power company supply far in advance of the service that they themselves can obtain from independent stations. This increases the cost of the service out of all proportion to the benefit that can be derived from such restrictions. The railroad companies frequently do not keep sufficient records of cost of power from their independent stations to enable them to determine the real cost of the service, and such figures as they do keep are usually shrouded in deep secrecy. In some cases, railroad companies, having independent power stations, later make connection with utilities and throw on the latter the service burden, power-factor correction, reserve capacity, etc., and then pride themselves that power from their own stations is cheaper.

An examination of Fig. 4 discloses the fact that the general shape of this curve would be very advantageous for combination with that of a utility with its maximum peak around 7 o'clock and its maximum hour between 8 and 9 o'clock. While the load itself is somewhat ragged and subject to violent fluctuation, it would be submerged in the daily load of a large utility with a good inherent system regulation. It is evident, however, that a power system with poor regulation would have difficulty in meeting the service demand of the railroad with a load of this character.

* (Parsons, Klapp, Brinckerhoff & Douglas), New York, N. Y.

Mr. Gibbs has stated, in part, as follows:

"Unfortunately, in the Eastern United States, available water powers are few, and such as exist are not generally near the consumer, and are often not dependable in capacity * * *."

This statement should not be applied to the entire Eastern United States because all through Northern New York and New England there are large districts which have excellent service from water power, and which are not normally supplemented by fuel-burning plants.

Mr. Gibbs also states, in part:

"As a matter of fact, the load conditions * * * are so exceptional that it is doubtful whether any existing local company * * * could afford to take on a load of this character, except by devoting special generators and transmission circuits to the railway supply * * *."

Was this entirely due to the deficiencies of the local utilities or was it partly caused by the desirability of single-phase generation?

Referring to the author's statement that "the alternating-current system is the nearest approach so far to a general one," the speaker does not think that any trend has developed toward any particular system. England, France, Holland, Belgium, Spain, Australia, South Africa, and South America are using direct current; Switzerland and Germany are using single-phase, alternating current; and Italy is divided between direct and three-phase current. European conditions do not apply to American railways and, in general, the street car and interurban systems in this country are using direct current at varying voltages, and such changes as have been made in existing systems, have all been from alternating current to direct current. The largest main-line electrification in this country is that of the Chicago, Milwaukee and St. Paul Railroad using direct current at 3 000 volts. The most recent electrifications (except that of the Pennsylvania Railroad) have been the Illinois Central at 1 500 volts direct current, and the Great Northern, with alternating-current distribution and direct-current locomotives. Most American terminal electrifications, including that of the Pennsylvania, have been at 750 volts direct current. On the other hand, the New York, New Haven and Hartford, the Pennsylvania, and the New York Connecting Railways are equipped with alternating-current distribution and single-phase motors; and, finally, the electrifications of the Norfolk and Western and the Virginian Railways are based on alternating-current distribution and contact wires and three-phase motors. It can hardly be said, therefore, that any system has been general, or that railroads are even approaching a general system.

The author states that the generators in the power-house are rated at 10 000, single-phase, continuous rating, and the speaker inquires as to what the equivalent rating of these generators would be if operated at three-phase. Was consideration given to the operation of motor-generator locomotives, or of single-phase motors instead of the split-phase motors, and, if so, what considerations led to their being discarded?

Mr. Gibbs has given an extremely interesting and clear description of the balancing wires and balancing transformers which are unique and useful

features of this system and for which he is largely responsible, if not for their actual invention at least for their practical application. The speaker would like to ask whether the balancing transformers, which are almost equal in total capacity to the main transformers, are not (to all intents and purposes) useful and necessary, and whether, in the event of their omission, it would not be necessary to put in equivalent capacity in the main transformer stations? Would it have been possible to space the transformer stations 20 miles apart if it had not been for the addition of these balancing stations?

In addition to these questions, the following are also submitted:

(1) What normal pressure is maintained between the trolley wire and the pantograph as used on the Virginian Railway?

(2) Mr. Gibbs states,

"Under these conditions the motors are subjected to high-current inputs and may be damaged unless they are of a type which will meet this severe requirement."

Can a single-phase motor be constructed that will meet these requirements?

(3) The speaker notes the connection of two driving wheels to a single motor. Is any difficulty experienced in the distribution of the load between the two motors included in each unit?

(4) The split-phase motor is limited to two speeds except when operated in connection with a rheostat. This would seem to limit its use to heavy freight traffic where a comparatively low rate of speed was sufficient and would not make it suitable for general railway operation under varying conditions. How much is the rheostat used in ordinary operation other than in acceleration?

(5) Rheostatic operation is presumably rather expensive in power consumption, and in this connection the speaker would also like to know how much the speed limitations influence the character of the load curve, since it would appear that a train might be required to go up a heavy grade at a higher speed than was absolutely necessary.

The speaker wishes to congratulate Mr. Gibbs on the accuracy of his preliminary power estimates and on the very brief but satisfactory statement of the general results.

R. D. N. SIMHAM,* ASSOC. M. AM. SOC. C. E. (by letter).—The author describes the broad engineering features of the railway electrification problem quite completely and concisely. There is little more that can be said except to make a few remarks as to the merit or usefulness of a few of the details mentioned. The paper must be of immense practical help to engineers who are engaged in designing such electrification projects, particularly the data given in Tables 3 and 5.

The electrification of the Virginian Railway is one more illustration of the economy that would be possible by changing the motive power from steam to electricity. The power then comes from a central common source; that is, the generating unit is large and comprehensive. That is the reason

* Town Planning Asst., Madras, India.

why, from the points of view of economy of power, equipment and operation costs, and efficiency, electrification projects invariably prove successful and beneficial both to the Company and the public. The author states that the Virginian electrification project has effected considerable savings by the changed form of motive power. Such conclusive evidence is very encouraging to engineers engaged in other electrification projects and should certainly stimulate engineering activities throughout the world in establishing, with confidence, electric plants in the place of steam, oil, or other forms of power plant.

So far as India is concerned, it has great prospects for developing very cheap hydro-electric power and already several undertakings of great magnitude have been started. Hydro-electric power should be cheaper than steam turbo-electric power, at least so far as the conditions of India are concerned, where good, cheap coal is scarce and natural water power is plentiful. Another advantage of water power is that it requires only the single transformation from the energy of water into electricity; whereas in steam turbo-generators a double transformation is required. The first transformation is to convert the latent energy of the fuel into heat energy as stored in steam. The next transformation is to convert the steam energy into electrical energy.

Besides the loss involved in the double transformation one of the great disadvantages of the steam-driven station is the large amount of space required for the boiler equipment and the condenser plant. In the case of the Virginian Railway plant (see Table 3), the turbine room occupies 11 440 sq. ft., whereas the boiler-room space is 13 454 sq. ft. That is, much more space is allotted to the steam-raising plant than to the generating plant. The ratio of the turbine-room space to the boiler-room space is as 100 to 118. It is usual to find the boiler occupying as much as three times the ground space required for the generators. Again—and this is especially important—boilers cannot be over-loaded 50% like turbo-alternators, so that, if advantage is to be taken of the overload capacity of the generators at any time, the necessary extra boiler capacity must be secured.

In countries like America and England there are, perhaps, several reasons for the slow development of railway electrification. As stated* by H. P. Gillette, M. Am. Soc. C. E.,

"Resistance to change, which usually is stronger in an old and powerful institution like the railroads than in a young and struggling one, probably has been the most serious single cause of delay; and as might be expected, it appears to have operated more effectively in England than in America."

In India, however, where railroad projects are just beginning to be developed, there is no reason for delay. All the present railway projects (which may be counted on the fingers of one hand) are capitalized by outside enterprise, and certainly the financiers would not start experimenting and would always postpone so radical a step as electrification in the hope that some other capitalist would bear the first expense of the experiment.

* *Engineering and Contracting*, February 20, 1924, p. 385.

There is great prospect, however, that the problem of electrification will be influenced by the growth of the great power systems in India. It is recognized that the cost of power is becoming more and more important, since civilization is requiring increasingly larger amounts of cheap power per capita. It will become still more important to utilize all possible sources of natural power, which is plentiful in India and is now going to waste. Cheap fuel is scarce and the cost of transportation is high. Therefore, this problem is becoming more important and more serious each year. The development of hydro-electric power would relieve the situation tremendously. To India, electricity is indeed the only practical source of economical power. It is inexhaustible, and should displace all the old forms of motive power.

The considerations that should generally be the basis of any power plant design are well illustrated in Fig. 4. The somewhat sharp peaks and sudden falls in the curve indicate that it must be very difficult to maintain economical operation of the station. Quite contrary to what might be usually expected in a railway load curve, the paper shows that, on two occasions within a period of twelve hours, the entire station load was taken off; but the sudden falls are not necessarily so serious in the economical running of the plant. The sharp peak loads, especially those of very short durations, are, however, very disadvantageous. For the sake of supplying power just for the abrupt peak loads, all the generators must operate and thus there is a great deal of idle power which is not being utilized. In other words, by underloading the plant, the average output would have supplied the whole of the daily demand had that output been constant. In this respect the author seems to think, for obvious reasons, that it is impossible to secure, in practice, any such uniformity of load; but if it is possible to relieve the station of such peak loads by drawing the excess power from a secondary source of accumulated power supply, or by creating some form of special boosters whereby temporary high drafts for power may be made, it would mean, for the Virginian Railroad plant, the running of not more than three turbo-generator sets at a time.

The problem of meeting the demands of excessive peaks that occur for short periods must be carefully considered. Assuming that such a thing is possible, the maximum demand of the generators would be reduced from 48 000 kw. to 30 000 kw. during the peak intervals. This would make it possible to shut down one generator throughout the day. It is with an idea of meeting the highest peak loads that the extra plant is made necessary. This extra plant is a measure of the increase caused by the capital charges for the running plant and incidentally for buildings and other equipment. From the data given by the author one may note that the average load for a normal day is only about 30% and on a maximum congested day not more than 43% of the full capacity of the generating plant. The effect of this low ratio and of allowing the extra plant to run idle is noticeable in the actual output in kilowatt-hours per pound of coal consumed. The station coal rate has averaged even less than 1¾ lb. per kw-hr. of net output, or about 14½% efficiency, which is poor.

In referring to the use of pulverized fuel, it must be said that, although combustion would be better and firing easier, the fuel cannot constitute the source of energy by itself. It would be useless unless there was practically an unlimited supply and a sufficient circulation of air. It is invariably necessary in every plant to make careful calculations and provide suitable arrangements for the supply of oxygen to the combustion chamber. Perhaps the provisions made on the Virginian Railway are adequate. The author has shown all the principal merits of location and planning of the Virginian Railway power plant in a way that should open the eyes of designers to the conditions that must be satisfied, in order that a project shall be a financial success. The choice of site and other details of planning are decidedly problems for the engineer as distinct from the designer and inventor of machinery and apparatus.

A noteworthy item concerning the transmission arrangement is the decision to secure a separate right of way intersecting with the railway only at the sub-stations. In places where such a plan is practicable without future trouble and other public inconveniences, direct transmission routes should always be adopted, as far as possible. As a matter of fact the economy effected by reducing the transmission circuit in this project is very great. The saving in 34 miles means nearly 770 tons of steel in the towers, 136 miles of transmission cables, and other incidental expenses.

The choice between direct and alternating current is a matter of the very foremost importance. This will depend on the distance to which the electric current is to be supplied and the purpose for which it is to be used. It seems to have been agreed that, where there are no other conditions affecting the choice of current, the high-tension alternating system is best for larger supplies. It is also a fact that transformers for alternating current offer great advantages over those for direct current. They are stationary, require perhaps little or no attention, and can be constructed for any amount of power; and their efficiency may be as much as 96 or 98 per cent. From the point of view of economy in the electric mains or conductors, however, three-phase current is best; next is direct current; and last, alternating current. For railway traction itself three-phase motors are the best and most economical.

At present (1928), there are about fifteen locomotives on the Virginian Railway. Assuming that they are all moving in one direction in a day of, say, 12 hours, the time headway per train would be 1 hour per locomotive. At an average speed of 20 miles per hour, this would give an actual space headway of 20 miles between any two trains. The ordinary spacing of sub-stations with a much lower voltage arrangement would be 20 miles. From Table 6, it is found that, at present, the maximum number of trains either way (say, eastbound) is five. This would mean presumably an actual headway of 2.4 hours per train. Taking the lowest average of 14 miles per hour, the actual space headway would be 29.6 miles, instead of 20 miles. To satisfy the latter requirements five main sub-stations would have been sufficient under ordinary conditions of close sub-station spacing; but in this case the operation is done at a very high voltage. Taking full advantage of the balancing wire system

it would have been sufficient to increase the sub-station capacity and reduce the number of main sub-stations.

Referring to the main operating costs, the results given by the author are very encouraging. Generally, by electrifying the road, only in a few items, such as cost of engines and maintenance of equipment, are considerable savings to be expected because these would be about 45% of the total expenses. A 30% saving in these items would mean nearly 14% reduction in the total operating cost; but the enormous increase in the capitalization and the very large incidental increase in the item of maintenance of way and structures (including the trolley and other contract system and the supporting structures) is the main handicap against electrification. Sometimes these larger expenses would go a long way toward neutralizing the 14% saving; but however small the savings may be, electrification is an up-to-date development and will displace other kinds of power in the future. It can perform a service beyond the possibilities of the steam engine and it can effect greater operating economies than are possible by steam propulsion, so that in the future these may be sufficient to pay a reasonably attractive return on the new capital charge incurred. This has proved to be the case in the Virginian electrification.

GEORGE GIBBS,* M. AM. SOC. C. E. (by letter).—The discussion is confined chiefly to comments on electrical and power matters. None has been made on the purely civil engineering details of design or on methods followed in the construction and application of the peculiarly electrical devices to the railway right of way. The writer hoped to have had suggestions from engineers who have the care of the physical plant of railways as to the effect of the presence of structures and appliances upon clearances and operating procedure. It is evident that the presence, everywhere, over or along the tracks of conductors designed to secure continuous physical contact by a device on the locomotive means an invariable location (within narrow limits) of these conductors with respect to the tracks and adequate supporting structures alongside. All these restrictions raise difficult problems of accommodation between right of way and electrical construction and it is important that they should be adjusted in the best possible manner between the interested parties in order to produce economy in first cost and maintenance.

The paper, presented to a Society which has not been especially interested in electrical devices, was intended primarily to stimulate interest and inquiry into a method of railway traction which is becoming increasingly important and which is, therefore, a matter of concern to engineers having to do with the design or care of every department of a railway. The writer hopes that, in spite of the absence of comment from members engaged in maintenance-of-way work, the paper will be found helpful to them and will assist in the further development of details best suited to the physical conditions and limitations of railway location.

While the contact devices of an electric traction system impose obstructions on the permanent way of a railway, electric operation permits steeper grades

* Cons. Engr.; Chf. Engr., Elec. Traction, L. I. R. R.; Cons. Engr., P. R. R., New York, N. Y.

than are allowable with steam haulage, especially in the case of motor-car passenger trains on which can be concentrated an almost unlimited motive power. Similarly, trains hauled by locomotives can be grouped for multiple control from one point when it is necessary to furnish increased tractive effort; in the case of steam traction the dual control of two locomotives is open to objection, as is well known.

Mr. Hogan has raised a number of interesting questions in connection with heavy electric traction systems and methods. Some of these are rather outside the scope of the paper, but may be answered briefly.

He is interested in the "system" controversy. This is not the place to enter into a discussion of the relative merits of different electric systems, for the reason that the paper was intended to cover only the considerations and conditions which led to the adoption of a certain system as best adapted for the Virginian Railway. He takes exception to the remark that the "alternating-current system is the nearest approach so far to a general one". The writer made this statement only in connection with a brief explanation of the trend of development of systems, beginning with appliances for light or comparatively short-distance haulage and leading up to those required for the heaviest character of trunk-line work over long distances. The distinction was made in order to show the reason for using high voltages in the transmission and the current-collecting devices at the trains for the latter services.

Some systems are limited in respect to the working voltage and are, therefore, most suitable for light or short-distance work; but the alternating-current system, being flexible as regards working voltage, is adaptable for any character of traction. The word, "general", used by the writer, has no reference to the extent of present usage of systems, which is really a factor of little significance in determining on a system for a particular case. Many of the installations cited by Mr. Hogan can be considered only as examples of the progressive steps followed in the development and application of heavy electric traction. Therefore, to draw any conclusions from a catalog of such installations it is necessary to take into account both the dates at which they were installed and the local railway conditions surrounding each. As matters now stand, there is no one system in exclusive use, and this is as it should be. Developments should be allowed to continue along different lines and by different systems before being tied down to a standardization on one, which would preclude further useful advances in the art.

This point has been well brought out in the discussion submitted by M. Japiot, who is one of the French railway officials responsible for the selection of a system for the French railways. He is a man of broad railway experience, a keen student of electric traction, and is unusually familiar with what has been done in the United States as well as elsewhere in this field. He recognizes that European railway conditions are quite different from those in America and may require different methods for their solution. He brings out the important point, moreover, that the need for standardization on one system is not as great in America as it is in Europeon countries, stating that,

"With the unrestricted liberty still prevailing in the American field, engineers in the United States have the opportunity to give each individual problem

the most favorable solution. * * * standardization must always be a compromise between the advantages and drawbacks of the different systems of electrification for several various problems."

He goes on to say that "if any compromise had to be made at an early date (which does not seem to be probable), the single-phase overhead line might be adopted successfully in America." The writer is not arguing that question now, and merely wishes to state that the reasons in favor of the system which was adopted on the Virginian Railway were controlling for that case. Furthermore, it can be safely stated that no one connected either with the design of the installation or its successful operation on that particular railway, has found any reason to regret the decisions made.

Mr. Hogan refers at some length to the consideration which led to the construction of an independent power plant by the Railway Company. The problem of power supply for the road was most carefully studied for the particular situation and character of load. All outside local sources of power were examined and it was found that no existing power company in that particular field was willing to submit a proposition which could be considered.

These definite facts, therefore, should dispose of the question for this particular case, and in stating this there is no intention of reflecting unfavorably on commercial power supply in general or in particular. As stated in the paper, "doubtless, there is a field for commercial power companies, especially large networks, in the supply of power for heavy railway traction, * * *".

As an example, the Pennsylvania Railroad Company has closed a contract on satisfactory terms with an independent power company for its present and future single-phase power requirements, which contemplate very large energy demands for an extensive electrification plan. This power company, the Philadelphia Electric Company, has a large generating capacity from a number of plants, both steam and hydro-electric, tied together in a great distributing network of lines. It was in an unusually favorable position to take on a railway load, which contemplates electrification of both freight and passenger services with a high daily load factor. None of these conditions applies to the case of the Virginian Railway.

Further, Mr. Hogan objects to the statement that "in the Eastern United States available water powers are few, and such as exist are not generally near the consumer, and are often not dependable in capacity." He thinks that this does not apply to the entire Eastern United States. This may be true for certain locations in the East and for near-by and, as a whole, rather limited power demands. However, the point is that the extension of electric traction generally should not be predicated on the availability of hydro-electric energy, and it is not necessary that it should. Water is often said to be a "free gift of Nature", but electric energy produced from it is far from being "free" and may, in fact, be very expensive at the point received. Doubtless railway electrification will grow in extent by reason of its advantages and economies, in certain cases, over steam haulage. It is from these viewpoints that it should be examined. Fuel-burning, however, to produce electric energy must, in the nature of things, be depended on for the majority of these enterprises.

Mr. Hogan asks a number of questions relating to details of the electrical apparatus of the Viriginian Railway installation. He refers to the capacity of the generator sets, stated as 10 000-kw., continuous, single-phase rating; the capacity of this generator would be greater, of course, if loaded three-phase. The turbine, however, is built to suit the load. Therefore, to state the case more accurately, if the output were three-phase the generator would be smaller and the power plant otherwise would be the same.

As to the type of motor used, Mr. Hogan asks whether the single-phase motor can be constructed so as to meet the requirements of this service. This is best answered by stating that on the Pennsylvania Railroad (where similar service as to grades and heavy trains and starting conditions will obtain on the Pittsburgh Division when electrified) the straight single-phase type of motor has been adopted, built, and tested, and the conclusion is that it will satisfactorily meet such service conditions.

There has been no difficulty experienced in the distribution of load between the driving-wheels that are coupled to the different jack-shafts of the Virginian locomotive units. Of course, there are times when one motor will slip its coupled wheels and the other coupled unit will not, but there is no difficulty in a practical way in starting and hauling trains on that account.

As to the necessity of using the rheostats in the handling of trains, other than during acceleration periods, this is, in point of fact, not a necessity and is a very small practical factor anyway. Occasionally, an engineman may cut in the rheostat descending light grades when he wishes to operate at more than 15 miles per hour (the normal speed being 15 miles per hour descending grades). In such case, by using resistance, the motors will operate above synchronism, and a speed of 18 to 22 miles per hour can be secured. Under these conditions almost no energy is returned to the line, either at 15 miles per hour or at the higher speeds.

The question is also asked as to how much the two-speed characteristic of the locomotive influences the system load curve. It does affect it to some extent, but the difference is not controlling in its effect on the general character of the load curve or in the power plant capacity. Other considerations, such as definitely limiting maximum running speeds to a safe figure in handling these very heavy trains over the division in the desired time, are of more importance. It is unsafe, in this as in many other matters, to generalize from an incidental factor in establishing a desirable over-all solution for a particular case. As to the normal pressure maintained between the trolley wire and the collecting device on the locomotive, each pantograph carries two shoes 4 in. wide having centers 10 in. apart; the pressure of each shoe against the wire is set at 10 lb. Answers to other questions submitted by Mr. Hogan will be found in Mr. Hill's discussion.

Mr. Potter speaks of the great desirability of "regeneration" in moving down heavy grades, and refers—aside from energy savings secured thereby—to the difficulties attending the use of air-brakes on these heavy trains at such times. He also mentions that when the issue is between the question of power demand and train movement, the latter is the important one, inasmuch as "transportation is the purpose of the railway". All this is true and well

expressed. In the paper, the writer mentions that the regenerated energy on the Virginian was found to be more than 8% of the total used to operate the road. The latest figures, covering 12 months' operation, show that the actual value was 9.1%, as taken from the watt-hour meters on the locomotives. This is obviously an important amount of energy saved, but does not express all the advantages secured by "electric braking". Safe and smooth control of the 9 000-ton train on the 12-mile Kellysville Hill (1.4% average grade) has been found to be a most important result.

Mr. Ray refers in an interesting way to the experience of his road, the Delaware, Lackawanna, and Western, in handling heavy freight trains by steam on a certain steep descending grade 17 miles long, where it is necessary to restrict movement to a speed of 15 miles per hour. This requires thirty-five brake applications and releases during the descent. In contrast to this complicated operation, J. W. Sasser, Superintendent of Motive Power of the Virginian Railway, states* that, under electric operation on his railway, brakes are only regularly applied once during the 131-mile run from Elmore to Roanoke (the entire electrified division). He also states that 100% increased life has been secured from brake-shoes over that previously obtained under steam operation, and that slide-flat wheels are of rare occurrence. It naturally follows that the car as a whole receives reduced wear and tear over the division, and that train delays, accidents, and derailments on account of defective brakes are rare. A prominent Western railway official, on a recent inspection of the Virginian installation, commented on the remarkably smooth handling of these electrically operated trains over an undulating division, and the absence of surges and jars in starting and while in motion. Also, the absence of jar in stopping, due to it being unnecessary to apply the automatic brakes, is noted; final stop is made with the straight air-brake on the engine only.

Mr. Litchfield refers to a contributing cause of smooth operation from the constant-speed characteristics of the locomotives. By riding on the trains observers cannot fail to be impressed with this result.

Mr. Semenza is interested in knowing the relative weight and cost efficiencies of different types of electric locomotives. This is, in the writer's opinion, really a question for determination at a future time. The effort on the Virginian was to adopt initially a locomotive of tried capabilities and of rugged construction. Other reasons for the locomotive design adopted are given in the paper. Refinements in type, details, and weight reduction will naturally follow further experience here and elsewhere. It would seem that foreign engineers have been more interested than those in America in developing locomotive designs to secure maximum weight efficiency. In the writer's opinion, some European locomotives show undeniable complication and a paring down of weights of parts to an extent which would be inadmissible under American railway conditions.

Mr. Öfverholm comments on certain differences between Swedish and American practice in the design of the distribution systems of electric railways and in the provisions adopted to prevent disturbances in near-by telephone and telegraph wires. The information he gives is interesting, but it is

* *Bulletin*, Am. Railway Assoc., June, 1928.

unsafe to conclude from the particular to the general for the preferred solution of complicated cases. Swedish electrifications are quite extensive and have been planned to conduct, successfully, traffic which in some cases is heavy compared with general European practice, but is always light as compared with American practice. It is especially very light as compared with the train loads dealt with in the Virginian Railway electrification. The treatment adopted by foreign engineers, therefore, for details can hardly be expected, in all cases, to fit in with American ideas and the radically different traffic conditions here.

Mr. Budd is undoubtedly correct in stating that "on most of the railway mileage of the country * * * traffic density is not great enough to pay a return on the cost of electrification sufficient to make the investment attractive". He refers to the constant improvement made in the efficiency of the modern steam locomotive, both in capacity and economy, which makes it difficult to show the same advantages for electric locomotive operation over steam as was the case some years ago. This is a true statement and emphasizes the importance in making a thorough investigation of all factors governing a proposed use of electric instead of steam operation as applied to the particular case in hand and the danger of drawing the conclusion that if electric traction has been found advantageous in one location it would be equally so in another. An analysis of the individual problem, therefore, should always be made beforehand and by competent authorities in railway operation generally and in the special technical features connected with the two methods of operation. The Virginian Railway Company was confronted with the necessity of a very extensive physical rehabilitation of its property in order successfully to conduct its rapidly increasing business, and it became a question of whether it would be better to secure the increased capacity by a continuation of steam haulage with expensive additions to the main tracks, to get the best obtainable road capacity from steam locomotives; or, on the other hand, to make its present single main track adequate for a long time by using electric locomotives of unlimited capacity at a greater running speed. Electing to do the latter, the Company obtained a form of motive power which insured maximum hauling capabilities and sufficient for present needs. It still has the possibility of obtaining, when needed, additional road capacity by double-tracking.

Mr. Budd's questions regarding the type of electric locomotive used are answered elsewhere. The Virginian is not tied down to any one particular type. If in the future it should be found advantageous to use another, it can be done without in any way changing the main and expensive features of the electrification, namely, the power plant, the transmission lines, and the distribution system over the tracks.

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Paper No. 1709

FLOOD CONTROL WITH SPECIAL REFERENCE TO THE MISSISSIPPI RIVER

A SYMPOSIUM*

PAGE

Introduction.
By Edgar Jadwin, M. Am. Soc. C. E. 657

Résumé of the Mississippi River Problem.
By C. McD. Townsend, M. Am. Soc. C. E. 659

Rainfall Characteristics of the Mississippi Drainage Basin.
By H. C. Frankenfield, Esq. 675

Run-Off Characteristics of the Mississippi Drainage Basin.
By Nathan C. Grover, M. Am. Soc. C. E. 690

The Work of the Mississippi River Commission.
By C. W. Kutz, M. Am. Soc. C. E. 697

Forest Cover as a Factor in Flood Control.
By E. F. McCarthy, Esq. 716

Reservoirs for Mississippi Valley Flood Protection.
By William Kelly, M. Am. Soc. C. E. 724

The Basis of the Case Against Reservoirs for Mississippi Flood Control.
By Arthur E. Morgan, M. Am. Soc. C. E. 737

* Presented at the Technical Meeting and at the meeting of the Waterways Division, Columbus, Ohio, October 12 and 13, 1927.

SYMPOSIUM (*Continued*): PAGE

Levees as a Means of Flood Control for the Mississippi River.

BY J. F. COLEMAN, M. AM. SOC. C. E..... 755

Improvement of Navigation in Relation to Flood Control.

BY STUART C. GODFREY, M. AM. SOC. C. E..... 762

The Flood Problem of New Orleans, Louisiana.

BY MARCEL GARSAUD, M. AM. SOC. C. E..... 782

The Relief Outlets and By-Passes of the Sacramento Valley Flood-Control Project.

BY C. E. GRUNSKY, PAST-PRESIDENT, AM. SOC. C. E..... 791

Reclamation as Affecting Flood Control.

BY ELWOOD MEAD, M. AM. SOC. C. E..... 812

Power as Affecting Flood Control.

BY F. W. SCHEIDENHELM, M. AM. SOC. C. E..... 815

The Effect of Agricultural Drainage Upon Flood Run-Off.

BY SHERMAN M. WOODWARD AND FLOYD A. NAGLER, MEMBERS, AM. SOC. C. E..... 821

WITH DISCUSSION BY MESSRS. J. P. SNOW, FRANK B. MALTBY, MORRIS KNOWLES, CHARLES H. PAUL, F. G. JONAH, C. S. JARVIS, EDWARD H. SCHULZ, Q. C. AYRES, H. S. GLADFELTER, BYRON E. WHITE, W. G. PRICE, OREN REED, HAROLD C. FISKE, CHARLES F. BROOKS, WILLIAM GERIG, C. H. EIFFERT, SHERMAN M. WOODWARD, W. J. SCHLICK, S. L. MOYER, WILLIAM M. HALL, C. E. RAMSER, T. KENNARD THOMSON, W. E. ELAM, R. W. DAVENPORT, E. W. LANE, C. MCD. TOWNSEND, H. C. FRANKENFIELD, E. F. MCCARTHY, WILLIAM KELLY, ARTHUR E. MORGAN, MARCEL GARSAUD, C. E. GRUNSKY, AND SHERMAN M. WOODWARD AND FLOYD A. NAGLER.

FLOOD CONTROL WITH SPECIAL REFERENCE TO THE MISSISSIPPI RIVER

INTRODUCTION

By Edgar Jadwin,* M. Am. Soc. C. E.

The importance of controlling the floods of the Mississippi River is now generally recognized. It is a National problem. The Mississippi Valley contains the largest area of rich soil in the country. It must be inhabited and used to as great an extent as practicable.

This valley is about 600 miles long and 50 miles wide. The entire area is generally considered to have been formed by the river itself. It is the natural bed of the river in time of flood. In its natural state it was flooded practically annually. This rendered intensive cultivation of the land too uncertain for practical purposes.

It has been compressed or narrowed in recent years to an increasing extent. This work has been done by levees, which have kept the water off the land so that it can be cultivated on an average of at least four out of five years. Some parts of the reclaimed land have not been flooded for much longer periods.

How much further can the river be compressed without having it break out at intervals and do even greater damage than it would have done if the works had not been constructed? The population in the valley is increasing and a crevasse is now of greater threat than formerly. In the further control of the river, to what extent shall levees, dredging, reservoirs, spillways, floodways, and other safety measures be used? Will it be possible to reclaim the entire valley permanently? Or must some part of it be permanently or intermittently dedicated to the service of the river?

In watching some piece of machinery, such as the steam shovel, probably every one has commented upon its likeness to a living thing. The steam shovel is guided by a man, expresses his aim, and has good reasons for appearing lifelike. The Mississippi River, without human aid, comes as near being lifelike in its own right as any inanimate object in the world. It consists of water and dirt aided by gravity. Some one has calculated that its volume in flood would fill the great Lake Erie in two weeks; some one else, that its energy when in flood would, if expressed in pounds and the square of its velocity, be measured in quintrillions. It tears the dirt from its banks and bed. Some one else has calculated the dirt carried annually as equivalent to a cubic mile in volume. A large part of this dirt was formerly deposited on the alluvial valley, re-fertilizing the ground and slightly raising it. The works along the river and at its mouth so concentrate the flow of this dirt that the mouth of the river is being extended into the Gulf of Mexico 5 miles per 100 years.

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In talking with those who have given the best part of their lives to the river, the speaker was impressed with their admission of the results obtained by levees and equally by the shake of the head when there is talk of raising them much higher. They know the power of the river, and they know that work to be successful must reckon with this power. They know the danger of the crevasse and that the higher the flood-plane and the greater the population behind it, the greater the threat of the crevasse. The first three papers presented herewith describe largely the history of the river, the work done on it, and fundamental data needed for the solution. In the other papers, various phases of the problem are considered. The authors may not all agree, but if one will follow through, he cannot fail to acquire a greater and broader comprehension of the subject.

RESUME OF THE MISSISSIPPI RIVER FLOOD PROBLEM

By C. McD. Townsend,* M. Am. Soc. C. E.

Introduction

The problem of flood protection is one which has vexed mankind for ages. In Genesis VI, 12, 20, 24, a flood is described, which was caused by a rainfall of 40 days and 40 nights and which raised the waters 15 cubits, covered the earth for 150 days, and destroyed all animal life in its path. This description of the Deluge would not be a very exaggerated account of the flood of 1927 in the Mississippi Valley. Moreover, there is an analogy between the Ark that Noah constructed as a means of flood protection and the boats and barges used in 1927 for the same purpose.

The Chinese appear to have records of floods extending over a period of 4 000 years, and "thirty-five years ago more than 1 000 000 people (some say 7 000 000) perished by drowning and starvation, resulting from one flood which came from a break through the south dike of the Yellow River, about 20 miles above Kai-feng City."† The annals of the countries of Europe also record disastrous overflows with a loss of life far greater than that from any which have occurred in the United States. The loss of life from floods in Europe during three months in 1927, largely exceeded the deaths in the United States from the same cause during the entire year.

The most fertile portions of the earth's surface are the valleys of streams that are periodically subject to overflow, and, therefore, enriched by the alluvium brought down and deposited from the mountains. This fertility induces a settlement of the country, and the farmer who first tills the valley is prepared to lose an occasional crop by flood, relying on the increased yield in other years to recoup him for his losses. In fact, in certain regions, the difference in yield between hill land and bottom-land is so great that the farmer reaps a greater profit by tilling the bottoms, even if he loses his crop every third or fourth year, as is demonstrated by the high prices that are paid for land along the banks of the Missouri River even when unprotected by levees.

The early settlers usually adopted Noah's method of flood protection. They built their huts on the high ground along the river bank, and constructed a boat into which they moved with their few household goods whenever the waters invaded their dwellings. It is the method used by the trappers and lumbermen living along the bayous in the Atchafalaya Basin at the present time.

A second method of flood protection adopted at an early date was that of building a mound of earth or of erecting a platform above the flood waters,

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† "Flood Problems in China," by John R. Freeman, Past-President, Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 1405.

on which the home was located. The numerous Indian mounds found in the Mississippi Valley were probably built for that purpose.

If it is only necessary to provide for the population, this method of flood protection is economical and efficacious, and is adapted to rivers which overflow their banks at periods of the year when vegetation is not injuriously affected thereby. Thus, in the Valley of the Lower Nile, irrigation is necessary to produce a crop. It is only after flood waters have covered the land that vegetation will grow. Its floods are caused by rains originating in the tropics, and occur early in the spring before the period of planting. The problem in Egypt is not to prevent floods, but to impound their surplus waters so as to utilize them for a further flooding of the land during periods of drought while the crops are growing. In the Lower Mississippi Valley conditions are reversed. The floods caused by rains falling in the temperate zone through which the river flows, inundate the valley after the crops begin to mature, and they will be frequently destroyed unless protected from the overflow.

The occasional loss of a crop or even the destruction of his habitation was of little consequence to the aborigine, but as civilization progresses man becomes more firmly bound to the soil. It was a natural development from the mound of earth on which the dwelling was placed to a ridge of earth surrounding the area that was being cultivated, and so long as the area of land which was being tilled was a small percentage of that of the valley, levees could be constructed cheaply by the property owner, which would protect his crops from ordinary floods. If he was energetic and far-sighted, he insured his property from even the occasional overflows, which overwhelmed the remainder of the valley, by building his levee a little higher than that of his neighbor.

As the population increased and a larger proportion of the river bed was used for agricultural purposes, a mysterious increase in the heights and frequency of floods was observed, which was attributed by scientists in olden days to the wrath of the gods and by those of the present generation to the cutting down of forests which are assumed to have heretofore prevented rainfall or held in place soil which is now raising the river bed. A much simpler solution of the mystery occurs to the engineer, that is, that the same quantity of water can only be made to flow through a contracted channel by increasing the head or height and thus creating a greater velocity.

In narrow river valleys with small flood discharges, this increase in flood heights is not excessive, but when it is attempted to remove from the Mississippi River about 30 000 sq. miles of its flood channel, and contract a flow over an original width exceeding 40 miles to a channel less than 1 mile wide, the problem is a serious one. The nations of Europe have solved the problem for such rivers as the Rhine, Seine, Loire, Rhone, Po, and the Danube under the conditions which exist in that region, but it is to be noted that the flood discharge of any of these rivers is approximately that of a large crevasse during the 1927 Mississippi River flood.

The Legal Status of Flood Protection

When the French under Bienvielle settled the Lower Mississippi Valley, they had a knowledge of the methods of flood protection then adopted in Europe, and the original grants of land contained a proviso which required the grantee to construct and maintain a levee line along the river-front of his property. This servitude on the land was maintained in Louisiana after it was admitted into the Union, and only ceased by reason of an amendment adopted by the State within the past five years, which transfers it to the levee district in which the land is located.

This servitude was not oppressive so long as the plantations to be protected were limited to those in the State along the river bank, as only an insignificant part of the river bed was being occupied. It was only after settlement extended to the neighboring States of Arkansas and Mississippi that an appreciable increase in floods was observed. The lands of these States were originally a part of the public domain, but the "Swamp Act" passed by Congress, September 28, 1850, ceded those subject to overflow to the States in which they were situated, with a proviso that the cession to the State was to enable it "to construct the necessary levees and drains to reclaim the swamp and overflowed lands therein." The States of Louisiana, Arkansas, Mississippi, and Missouri organized offices for the sale of the swamp lands, and appointed commissioners for the location and construction of levees.

There had already been an appreciable increase in flood heights, which led to the passage of an Act approved September 30, 1850, directing "a topographical and hydrographical survey of the delta of the Mississippi River with such investigations as may lead to determine the most practicable plan for securing it from inundation", etc. The results of this survey were published in the report of Humphreys and Abbot on "The Physics and Hydraulics of the Mississippi River." This immortal work contains a scientific analysis of nearly every suggestion of flood relief which is being advocated at the present time (1927), and when the limited data which were then available are taken into consideration, it is a marvelously accurate forecast of the work necessary to protect the valley securely. Fig. 1 shows the average dimensions of the levee that existed in the vicinity of Arkansas City, Ark., in 1882, twenty-one years after they submitted their report, the levee profile recommended by them, and that suggested by the writer in 1893, and practically adopted by the Mississippi River Commission in 1913.

During the Civil War, the levee lines were cut by the contending forces, and seriously damaged by floods. After the Civil War local levee boards generally replaced the State organizations, originally appointed under the Swamp Act, although in Louisiana, a State Board of Engineers continued to exercise supervision over the local levee boards which had replaced the riparian owner in levee construction. Although the construction of levees by local boards gave more satisfactory results, than when the riparian owner was the sole judge of the dimensions, this method of construction was also objectionable. While one district was constructing levees of relatively large size, a neighboring district, either from a lack of funds or from erroneous

views of the height which floods would attain, would provide a weaker section, and crevasses would occur in levees beyond the jurisdiction of the more provident residents, which would overflow their lands.

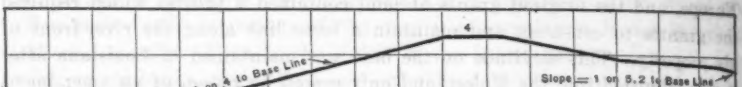

FIG. 1.

By an Act of Congress approved June 22, 1874, a commission was appointed to "make a full report to the President of the best system for reclamation and redemption of said alluvial lands from inundation." By an Act approved June 28, 1879, the Mississippi River Commission was created, and the duty of completing the survey authorized by the Act of June 22, 1874, was transferred to that body, and it was also directed "to take into consideration and mature such plan or plans, and estimates as will correct, permanently locate and deepen the channel, and protect the banks of the Mississippi River; improve and give safety and ease to the navigation thereof, and facilitate commerce, trade and the postal service." The Act also contained a proviso that "the Commission shall report upon the practicability, feasibility, and probable cost of the various plans known as the jetty system, the levee system, and the outlet system, as well as upon such others as they deem necessary."

The Act of March 3, 1881, making appropriation for the work of the Commission contained, however, the proviso that "no portion of the sum hereby appropriated shall be used in the repair or construction of levees for the purpose of preventing injury to lands by overflow, or for any purpose whatever except as a means of deepening or improving the channel of said river."

The Act of March 13, 1892, modified the wording of this proviso and directed that the funds appropriated "shall be expended for the general improvement of the river, for the building of levees and for surveys, * * * in such manner as in their opinion shall best improve navigation and promote the interests of commerce at all stages of the river."

The Act of March 1, 1917, contains a proviso that "no money appropriated under the authority of this Act shall be expended in the construction or repair of any levee unless and until assurances have been given satisfactory to the Commission that local interests protected thereby will contribute for such construction and repair a sum which the Commission shall determine to be just and equitable, but shall not be less than one-half of such sum as

may have been allotted by the Commission for such work." The same Act extended the jurisdiction of the Commission over levee construction to Rock Island, Ill. The Act of March 4, 1923, further extended its jurisdiction to the tributaries and outlets of the river so far as they may be affected by its flood waters.

It will be noted that in all these Acts of Congress while the authority of the United States over levees and its financial assistance has been progressively increased, there has been considerable care exercised to preserve a servitude on the lands which are to be benefited. As Comptroller General McCarl has stated, "the primary obligation for protection of private property adjacent to the streams for which co-operation between local interests and the Government has been provided, rests upon the individual States or subdivisions within which the private property is located, the Government's part being to assist financially in connection with the maintenance and development of the river as a navigable stream only in proportion to the amount made available by such State or subdivision."

Under this construction of the law, the question of servitude becomes the most important problem which Congress must consider in any project for an extensive development of flood protection. The States have transferred it from the riparian owner to the various levee districts. Shall it now be assumed by the State or by the Federal Government? It is evident from the results of the 1927 flood, that the cost of any project for absolute security will far exceed the capacity of the riparian owners to meet their portion of the expense provided by existing law. It is also apparent that the farmer tilling the less fertile hill lands of Arkansas and Mississippi will never consent to an increased taxation for the purpose of further increasing the value of the plantations in the delta region, and insuring the maintenance of such values.

The Mississippi River Commission

It will be noted that while Congress in the Act creating the Commission directs it to investigate various methods of flood protection, yet in the Act appropriating funds to carry on its work, it is expressly prohibited from using them for the purpose of protecting land from overflow. This apparent inconsistency in legislation had its effect on executive action and, therefore, merits explanation.

Most levee engineers in the Mississippi Valley had not accepted the conclusions of Humphreys and Abbot that levee construction would largely increase flood heights. There was a general belief that the concentration of the flood discharge would enlarge the river bed and that this enlargement would materially reduce the levee grade, and would greatly improve low-water navigation. These views had Congressional support, and as General Humphreys was then Chief of the Corps of Engineers, U. S. Army, it was considered advisable to appoint a commission of which the majority would not be connected with that Corps, and thus insure a more liberal policy in determining the effect of levees on channel improvement. As a majority of Congress

did not approve of appropriating public funds for the protection of private property, it was considered advisable to limit the powers of the Commission thus created by the proviso of the Act of 1881.

As a majority of the Commission which was first appointed believed that levees would improve low-water navigation, a liberal allotment of funds for levee construction was assured, but the proviso mentioned rendered it necessary that the levees thus improved should be located where they would produce the greatest influence on the low-water channel, instead of where they would best preserve the land from overflow. As low-water navigation was notoriously bad in the upper reaches of the river, there was a tendency to concentrate the allotments for levee construction along those sections, and particularly in the sparsely populated levee districts where the inhabitants did not have sufficient funds to protect themselves. A great impetus was given to levee construction, and a marked increase in flood heights resulted, so that not infrequently the protection afforded to the weaker districts caused crevasses in those more densely populated.

Levee construction from 1881 to 1891 not only largely increased flood heights, but it also failed to produce any appreciable improvement in low-water navigation. The Commission then investigated the effect of dredging on river bars, and came to the conclusion that a low-water navigation of 9 ft. could be obtained temporarily by this means and could be maintained more economically than by either levee construction or bank revetment. Congress then authorized the construction of dredges for that purpose, and permitted the allotment of funds for levee construction whenever it would facilitate high-water navigation of the river. Federal funds were allotted for levee enlargement under this proviso, and the weaker levee districts continued to derive a greater relative benefit.

The Act of 1917 caused a radical change in the methods of making allotments by providing that at least one-third the cost of levees constructed by the United States should be borne by local authorities. Under this method of making appropriation the more populous districts became the greater beneficiaries, as they could more readily raise the funds required to obtain Federal assistance. The weaker districts had difficulty in keeping pace with their more fortunate neighbors. The result is clearly shown in the record of crevasses during the recent flood, which most vividly emphasizes the Biblical saying that "unto every one that hath shall be given, and he shall have abundance; but from him that hath not shall be taken away even that which he hath." The levee districts which escaped overflow owe their immunity to the poverty of their less fortunate neighbors.

In compliance with the provisions of the organic act, the Commission has made extensive surveys and investigations for the purpose of determining the practicability, feasibility, and relative cost of the various methods of flood protection that have been proposed. The results of its investigations are scattered through its annual reports for the past forty-seven years, and have been summarized in papers and reports submitted by the writer from time to time. The gist of its conclusions was invariably that levees afforded the

most practical, feasible, and economic method, although its processes of reasoning varied with changes in the membership of the Commission.

So long as a majority believed that only an increase of flood height of 3 ft. should be provided for, which was the provisional grade for many years, the question was hardly worthy of serious argument. The crudest estimates of cost conclusively demonstrated that levees were so much cheaper than either reservoirs, the diversion of tributaries, or spillways, that a further discussion of the practicability or feasibility of these methods was merely an academic compliance with the law. It was only after a majority of the Commission endorsed Humphreys' and Abbot's views, and attempted to make provision for the greatest flood which had then occurred, that a careful analysis of the various methods of flood control became imperative.

After the flood of 1912, such an attempt was made. The Commission assumed as a major premise that the flood of 1882 was the greatest to which the levee system would be exposed, but based its estimates on the flood of 1912 which, while of shorter duration, had the same estimated maximum discharge, and had been more accurately gauged and measured. The Commission unanimously agreed that levees were the most economical method of flood protection, and that reservoirs were impracticable; but there was such a diversity of opinion with regard to the feasibility of spillways and the diversion of tributaries that the discussion of these subjects was omitted from the report. The views of individual members, however, were printed as Commission documents.

Effect of an Increase in Discharge on the Conclusions

When Humphreys and Abbot submitted their project for the protection of the Mississippi Valley from overflow, they based their computations on the flood of 1858 which was the largest flood through the entire valley of which there was then any record, although the flood of 1844 exceeded it above Cairo, Ill., and that of 1785 had similar characteristics to that of 1844. As stated, the Mississippi River Commission based its estimates of levee grades on the flood discharge of 1882, which largely exceeded that of 1858, and has only been exceeded since 1785 by that of the flood of 1927.

While both Humphreys and Abbot and the Mississippi River Commission came to the conclusion that levees were the most practicable and most economical method of flood protection, they based their argument on an assumed flood discharge, and it by no means follows that such flood discharges as are now being discussed as possibilities can be more cheaply controlled by this method. Any project which contemplates increasing the heights of existing levees at certain localities about 15 ft., not only affects the item of cost, but raises the question of the practicability and feasibility of maintaining such a levee line after it has been constructed. Whenever the Mississippi River flows in a channel 30 or 40 ft. deep, it has a powerful erosive force, and to expose levees in concave bends or across points to such a force would invite their destruction.

The cost of the existing methods of levee construction increases as the square of the height, and, if absolute protection is demanded for such levees as indicated, the cost over existing methods will probably approach the cube of the height. The cost of a weir is independent of the height of the water flowing over it, and the cost of a dam is approximately a direct function of the quantity of water which it impounds. If possibilities instead of probabilities are to control future levee construction, the cost will become so enormous that the economic solution of the problem will be found in other methods of flood protection.

The problem of flood control by reservoirs has been much simplified by the record of rainfall during the flood of 1927. It is self-evident that a reservoir which will reduce flood heights must be constructed in a locality where it can receive the waters which create the flood. This eliminates from consideration all reservoir sites in the Missouri Valley above Kansas City, Mo., in the Mississippi Valley above Rock Island, and in the Ohio Valley above Pittsburgh, Pa., as the rainfall in those regions did not exceed a total of 6 in. during the months of March and April. It practically limits their location to those portions of the States of Illinois and Indiana which drain into the Wabash River, the streams of Southwestern Illinois and Southeastern Missouri which discharge into the Mississippi River below St. Louis, Mo., the drainage area of the Tennessee River below the Wilson Dam, and the rivers of Arkansas, Mississippi, and Louisiana which empty into the river below Cairo. In this area there was a rainfall exceeding 12 in. in 60 days. The prairie country of Illinois, Indiana, and Southeastern Missouri is not adapted to the construction of large reservoirs. The flood of 1927, therefore, clearly demonstrates that the only reservoir or reservoirs capable of controlling the waters which created it must be located in the delta region below Cairo, that is, in the area which it is desired to protect by their construction.

It is theoretically possible to create in either the St. Francis, the Yazoo, or the Tensas Basins reservoirs which would have sufficient storage capacity to retain the surplus waters of the flood of 1927, but the cost would be excessive. Humphreys and Abbot suggested the utilization of the Tensas Basin to reduce floods, and the Mississippi River Commission has had before it propositions to utilize the St. Francis Basin for the same purpose. In the days of Humphreys and Abbot, the greater portion of the land needed for such a purpose was a part of the public domain and could have been acquired at a nominal cost. When the utilization of the St. Francis Basin was being considered, the State authorities were having difficulty in collecting the nominal taxation levied upon it. Levee construction, however, has enormously increased the value of property in both basins, and it is extremely doubtful whether the area for the reservoirs could be acquired by the Federal Government at the present time for $100 per acre, when the cities and villages which have sprung up in this region are taken into consideration, notwithstanding the reduction in values of real estate which must have resulted from the overflow.

The fact that it is estimated that more than 600 000 were made homeless by this flood and that it is one of the great catastrophes of the century is striking proof of the development of the delta from levee protection. Other floods have overflowed a greater area, but it was then so sparsely settled that the disaster did not create much comment beyond the Valley. If, to the cost of condemning about 6 000 sq. miles of reservoir sites, is added the cost of embankments for retaining the water higher than the levees which would be required, reservoirs can not be recommended as an economic proposition.

With spillways, conditions are reversed. For years, the writer has maintained that the existing Commission grade for levees below the mouth of Old River is the limiting line for their economical use. In a paper on spillways* presented before the Louisiana Engineering Society in 1925, he invited attention to the fact that a reduction in the flow of the Atchafalaya Outlet by the closure of Old River would necessitate the construction of a spillway to compensate for the increased discharge down the main river which would result therefrom. He also advocated the construction of a spillway below New Orleans, La., which would reduce flood heights at that locality, considering it a cheaper solution of the overflow problem for that city than raising its docks and the walls of the Industrial Canal to the Commission grade. He was then discussing the problem of lowering flood heights about 1 ft. The question ceases to be open to discussion, when it is proposed to provide for a discharge in the lower river 30 or 40% greater than that assumed by the Commission.

Although favoring a spillway which spills the water, and allows it to flow unrestrained to the Gulf, the writer also invited attention to the danger which arises from contracting the flow. The spillways required to reduce the discharge now considered possible to that assumed by the Commission, must have a flow exceeding that of the Yellow River in China or that of the Po River in Italy. The proposed flow in the spillway will exceed that of the South Pass of the Mississippi River, and approximately equal that of Southwest Pass, or Pass à Loutre, the other mouths of the river. Both South Pass and Southwest Pass have been successfully deepened by confining their flood discharge between parallel jetties. There are, however, hydraulic engineers who assert that the same discharge can be confined between parallel levees in a spillway without producing scour. It is a curious coincidence that the width proposed after the flood of 1922, for a spillway emptying into Lake Borgne, corresponds to the distance between the shore ends of the jetties at Southwest Pass recommended by noted levee engineers and adopted by Congress as the original project for its deepening to 35 ft., although a Board appointed by the Chief of Engineers in 1916 reduced this width from 6 000 ft. to 2 400 ft. because the excessive width as first approved was unable to maintain a channel depth exceeding 25 ft. The slope in Southwest Pass during floods is about 0.2 ft. per mile, while that in the proposed spillway would have exceeded 1 ft. per mile.

* *Proceedings*, Louisiana Eng. Soc., Vol XI, 1925.

A deep hole is invariably excavated in the contracted section of a crevasse, and the material thus scoured is deposited approximately on the arc of an ellipse enclosing the hole, and at a distance from it determined by the reduction in velocity caused by the dispersion of the flow. This action is similar to that which creates a bar at the mouth of the river. The forces which create the hole at a crevasse will exist in a spillway, and if not reduced by allowing its water to expand will be propagated its entire length. Some accidental cause, as an abandoned ditch or even a line of fence posts, will induce a local scour and when such a channel starts, it will enlarge. By such means both the Po and Yellow Rivers have created channels through which the entire river now flows.

In providing for possibilities which have not yet arisen, it is rash to ignore probabilities which have already occurred. The diversions of the Po and Yellow Rivers are historic facts while the excessive discharges now being discussed may be mere flights of a vivid imagination. The extension of the spillway at Point La Hache a sufficient distance up stream, will produce the same lowering of flood heights at New Orleans as the one proposed at Poydras, and at less cost. It will also reduce to a minimum the danger of river diversion.

An Act of Congress approved April 17, 1926, provides,

"* * * that the Secretary of War be and he is hereby authorized and directed to cause a survey to be made, and estimates of the cost of such controlled and regulated spillway or spillways as may be necessary for the diversion and control of a sufficient volume of the excess flood waters of the Mississippi River between Point Breeze and Fort Jackson in Louisiana in order to prevent the waters of the river exceeding 16, 17, 18, 19 and 20 ft. on the Carrollton gauge at New Orleans, and of approximately 46, 47 and 48 ft. on the gauge at Simmesport on the Atchafalaya Outlet. * * *"

Section 2 of the Act, however, contains a proviso "that no spillway shall be constructed as a result of the survey authorized by this act whereby the waters of the Mississippi River would be diverted into Mississippi Sound". A Board of Engineer Officers has been appointed in accordance with the terms of this Act, and will presumably submit its report before the next session of Congress. The Congress could never have contemplated the occurrence of the flood of 1927 when it passed the Act, and a strict compliance with its provisions under existing conditions would thwart its intent.

The river attained a height of 15.9 ft. on the Carrollton gauge during the flood of 1867, and of 46.3 ft. on the Red River Landing gauge. The levee line had been neglected during the Civil War, and practically annihilated by the floods of 1862 and 1867. A continuous spillway existed on both banks of the river from Baton Rouge, La., to its mouth, with the exception of the area occupied by the City of New Orleans. To assume that any spillway can be constructed in a semi-controlled river having a discharge at least 50% greater than that of 1867 which will only raise the gauge at Carrollton 0.1 ft. is absurd. Moreover, the proviso that no spillway constructed under the law shall divert water into Mississippi Sound precludes the construction of any spillway discharging into Lake Pontchartrain or Lake Borgne, and sug-

gests as a solution of the problem, a further extension up stream of the spillway recently built below Point La Hache, where the escaping waters flow into Breton Sound. The removal of the entire levee line on the east bank of the river below the Lake Borgne Canal could not be expected to reduce flood heights at Carrollton much below 20 ft. with a river discharge as great as has been recently estimated.

The gauge at Simmesport, La., can only be maintained at 46 ft. by closing Old River and preventing the discharge of the Mississippi River from entering it, and even then the existing reservoir at the lower end of the Tensas Basin will have to be maintained to reduce the maximum discharge which it is assumed flowed in the Ouachita and Red Rivers during the 1927 flood. The provisions of the Act which limits the survey to that part of the river between Point Breeze and Fort Jackson, La., also prevents the Board from investigating what gives promise of being the most satisfactory solution of the theoretical problem which is now being considered.

The disastrous results of the flood of 1927 are not primarily due to the floods of the Ohio and Upper Mississippi Rivers, but to the rainfall of 18 to 25 in. which occurred in the Lower Valley during the flood. Early floods in the rivers of Arkansas created such stages in the main river from Helena, Ark., to Vicksburg, Miss., that it destroyed what is termed the river's reservoir capacity, so that later moderate floods from the upper rivers created gauge heights through this section far exceeding any that had been heretofore recorded for the same discharge.

When the flood of 1927 is compared with that of 1912, on which the Commission based its estimates, it is noted that at Helena a discharge 300 000 sec-ft. less in 1927 than in 1912 increases flood heights 2.6 ft.; at Arkansas City, a reduction of 300 000 sec-ft. is accompanied by an increase in flood heights of 5.5 ft.; while, at Vicksburg, the same flood discharge increases flood heights 6.9 ft. When in addition to this disastrous condition of affairs, the crests of the floods of the Arkansas and White Rivers poured their waters on the crest of the main river flood, the question arises whether the cheapest solution of the problem would not be to separate these rivers entirely from the Mississippi and provide a separate channel for them across the Tensas Basin.

The Commission estimated that these rivers would only add to the maximum flood discharge of the Mississippi about 200 000 sec-ft. at their mouths, relying on the reservoir capacity of the main river and the lack of coincidence of the crests of the floods to reduce their reported maximum discharges to this quantity. If provision must be made for adding about 1 000 000 sec-ft. to a river the reservoir capacity of which has been exhausted, the grade of the levee line becomes excessive. If these rivers were diverted the reservoir capacity of the main river would be much greater than that estimated by the Commission and the flood of 1927 would have been propagated through the lower river, without the necessity of building spillways even if Old River was also closed. The cost of the proposed levee enlargement, and of spillways, would largely exceed the cost of such diversion.

It must be admitted, however, that this solution of the flood problem will not reduce flood heights at Simmesport to 46 ft., and that the determination of flood heights along the diversion channel would be a problem difficult of solution for the next twenty years, during which time the diverted flow would be excavating its bed. While the cost of this project may be many hundred million dollars less than that of any other project that will afford complete protection from any possible flood, it is still so great as to raise the question whether the increase in value of the land to be benefited would justify the expenditure.

The practicability of flood control by reforestation, by straightening the river, by enlarging its low-water channel with dredges, and by increasing its reservoir capacity by placing the levee lines at a greater distance from the channel, is not affected by a variation in the discharge. The only one of these propositions that is now being seriously considered is a further increase in the reservoir capacity.

For every mile that the levee line is moved away from the river bank, the reservoir area during floods will be increased by about 1 000 sq. miles, but the area capable of being cultivated will also be reduced by the same amount. The land along the concave banks of the river is the highest land in the delta, the most highly cultivated, and the location selected for the houses and barns of the inhabitants. Many levee districts would be bankrupt by a change of 1 mile in the location of the levee line. A large proportion of the productive land which now pays the taxes, would be left unprotected. While the Commission places levees in bends at a considerable distance from the bank, it is solely to protect them from river caving and insure a levee life of about twenty years.

The practicability of the project also merits consideration. If the flood of 1927 is to be an annual occurrence, it would adjust its channel to its discharge, but if it does not again occur during the next 100 years, the minor intermediate floods will tend to build up the river banks just as they have for ages, and by the time the second flood occurs, this fill will have largely reduced the reservoir depths.

It has also been suggested that complete protection necessitates a change in the standard levee section adopted by the Commission. In a discussion of the paper by the late William Starling, M. Am. Soc. C. E., entitled "The Discharge of the Mississippi River", presented to the Society in 1896,* the writer stated that,

"* * * the forms adopted for the levee section differ materially from those which obtain in reservoir construction. The causes of this difference arise from the impracticability of finding in an alluvial valley the foundation impervious to water that is an essential condition in reservoir construction. The material for constructing the puddled core is also not readily obtainable, and its value is not so evident when it can be flanked by water passing through a permeable foundation into the embankment beyond.

"The essential condition of levee construction is a mound of earth of sufficient height so that water cannot flow over its top; of sufficient mass so

* *Transactions*, Am. Soc. C. E., Vol. XXXV (1896), pp. 341–343.

that surfaces of saturation cannot be formed through it on which the superincumbent material will slide; of sufficient width of base, so that the water flowing through the permeable foundation will not have sufficient force to remove any material, it being assumed that the earth has been placed with care so that there are no channels left across the embankment through which the waters can directly flow. This mound of earth must be protected from the eroding action of rains, and its river surface from erosion by river currents and waves during floods. The grade must be somewhat higher than the highest flood to afford protection from wave action during storms.

"With the extreme variations in permeability which are found in the soils of which levees are constructed, variations in form become necessary, but practical experience with the soils of the Mississippi bottoms has developed certain forms which are of general application. A slope of 1 on 3, when well sodded with Bermuda grass, has been found to resist the action of rain, of river currents, and of minor waves. In localities exposed to the full force of waves through long reaches of the river, it is necessary either to reinforce the sod with some more permanent form of revetment, or to adopt a gentler slope.

"A width of crown varying from 6 to 10 ft. generally exists. The width to be given to the crown is interdependent with the slope to be given the land side of the levee. An 8-ft. crown affords convenient space for patrolling the levee line during floods, and storing such material as may be required to repair any damage which may be inflicted by storms. With a width of crown of 8 ft., a land slope of 1 on 2 can be safely employed until the levee attains a height of 6 ft.; with higher levees unless the soil is especially adapted to levee construction, there is a tendency to form a surface of saturation through the mass on which the upper portion is liable to slide. Levees not exceeding 12 to 14 ft. require a land slope of 1 to 3; above these heights in the upper basins of the river, trouble begins to be experienced with the foundation, and still gentler slopes are necessary, or the same result can be attained by adding to the base a mound of earth termed a banquette. * * * While these levee sections have resulted from practical experience rather than from theoretical considerations, it may be noted that the sections most generally employed in recent years insure an angle of repose of dry earth on wet clay of 14° when water is at the top of the levee, the limiting angle of such materials."

From these suggestions by the writer was evolved the standard levee section of the Commission (Fig. 1) which has been used on a levee line more than 1400 miles in length during a period of more than 30 years, and without a failure from weakness of section until the flood of 1927. During this flood, a disastrous crevasse occurred at Mounds, Miss., in a levee of standard section. A succession of rains converted the earth composing the parts of the levee above the banquette into a mud which could not withstand the static pressure due to a head of water of 6 ft., and it is natural to assume that the material placed in that part of the embankment was unsuited for levee construction.

When the writer was Assistant to the Commission and found material of such a character, he did not hesitate about enlarging the levee section which he had recommended. Such a course of action was so self-evident that he did not even consider it necessary to obtain the approval of the Commission for this change in form. It is illustrative of the hysteria that is sweeping over the country as a result of the flood that it is now proposed to prohibit the Assistant from exercising such a discretionary power, and to enlarge the

entire levee line to a section which will resist the water pressure, even if composed of poor material, notwithstanding the fact that more than 1 400 miles of levees of standard section, or weaker, successfully resisted the flood of 1927. This proposed reform will cost about $40 000 000.

Before making such an expenditure, however, it is advisable to make a further investigation of the causes of the conversion of this levee into mud. Several other levees, while not failing, showed similar signs of weakness, but it is a peculiarity of this flood alone. In all previous floods a mound of earth 6 ft. high, with an 8-ft. crown and slopes of 1 on 3, properly compacted and covered with a layer of Bermuda sod, could resist the water pressure no matter how porous the soil might be, of which it was composed, and if not protected by a revetment of Bermuda sod, rains eroded its slopes, instead of soaking into the mass and destroying its cohesion. During the flood of 1892, the writer, who was then Assistant to the Commission, had to hold a levee line 40 miles long in the Tensas Basin in Arkansas, a great part of which had a crown of 4 ft. and slopes of 1 on 2, not only until the water attained the top of the levees, but, by means of sand-bags and other devices, to still greater heights, and the only material that was converted into mud by rains was the fresh earth used in topping.

The Commission has passed strict regulations about allowing vehicles on the levee crown. If, however, during the excitement of flood protection, employees ignored those regulations and used automobiles for inspection purposes, the Bermuda sod would be rapidly destroyed during rainy weather, and longitudinal ruts formed from 6 in. to 1 ft. deep. With a 14-in. rainfall in 24 hours, these ruts would become veritable reservoirs for retaining water which could seep into the levee, and convert the best material into a mud. The excessive sloughing of the land slope of levees during this flood may also be due to this cause.

While the writer has no knowledge that such was the practice, before expending $40 000 000 on the levee line to enlarge the section, he would try the experiment of giving steep slopes to the crown, which would not only facilitate the run-off of the rainfall, but render it physically impossible for automobiles to violate the Commission's regulations. It is noted that the various projects for enlarging the levee section invite a greater use of the crown for travel in vehicles by widening it.

Conclusion

Congress has been extremely liberal in its appropriations for the Mississippi River since the creation of the Commission, but any one who assumes that Congress while limiting the expenditures for all the other rivers and harbors in the United States to $50 000 000 per year, can be persuaded to appropriate for the Mississippi alone $50 000 000 to $100 000 000 per year during a period of 10 years, has little knowledge of the workings of the Congressional mind.

Attention has been called officially to seven great projects of water development which will cost from $1 500 000 000 to $2 000 000 000, and there are numerous other expensive river and harbor projects which have local support. The American people will most liberally contribute a vast sum to the

Red Cross for humanitarian purposes, but when it is a question of expenditure from the United States Treasury, every community will demand consideration of its local needs. An equitable distribution of the funds over the entire country is necessary to insure the passage of a river and harbor bill.

The Mississippi River Commission has always been embarrassed by a lack of funds, and the fundamental principle governing its actions during the past 45 years has been to benefit the people of the delta as much as was possible with the appropriations made by Congress and under the limitations prescribed by that body. It is probable that it will have to pursue the same course in the future. If the attempt to establish a grade line which would be above all probable floods, as recommended by the writer when he was President of the Commission, should be abandoned, and the original Commission grade of 3 ft. above the stages that have occurred at the different stations along the river be substituted for it, there would be a probability of carrying out such a program with the funds which can be anticipated. Such a project not only would meet the economic situation, but would be in accordance with normal engineering practice.

The city engineer who designs sewers to carry off the rainfall has to solve the question, "What is the maximum run-off for which provision must be made?" The *Transactions* of the Society contains a number of discussions of this subject, and curves have been constructed showing graphically the probability of run-offs extending to periods of several hundred years. It is, however, generally recognized by the Engineering Profession that provision of a sewer system for a city, which will prevent the flooding of the streets under the most adverse conditions, is an unwarranted extravagance. For example, if the City of New Orleans had constructed a sewer system to provide for the rainfall which occurred during the recent flood, it would have been compelled to expend on it, funds which could have been more profitably diverted to street pavements, to a water supply, and to other essentials to a city's progress. The occasional over-taxing of the sewer system is less injurious to the city's development than the curtailment of other prime necessities.

When the problem is expanded from drainage of a city to that of a continent, its complexity increases and the determination of future possibilities becomes more uncertain. Such possibilities are not then determined by mathematical computations, but by the imagination. When it is stated that a flood like that of 1927 will not probably occur again in 100 years, the engineer must take into consideration probabilities instead of possibilities, and consider the interest on investment for a period of 100 years. If the interest is computed at 4% on the cost of some of the proposed projects, it would be greater in 15 years than the estimated damage caused by the flood. A disaster which will not probably occur once in 100 years is in the same category as an earthquake or cyclone. It is an Act of God for which the philanthropist should make provision. Because an earthquake formed Reelfoot Lake opposite New Madrid, Mo., 100 years ago, engineers do not make the buildings in the Mississippi Valley earthquake-proof.

It is suggested, however, that the engineer who attempts to solve the problem must make a definite decision whether he will be governed by probabilities or by possibilities. To submit a project which in one part provides only for what will probably occur, while in another makes provision for a possible increase, will be trying to serve two masters, with the usual result.

The final suggestion is also submitted that no matter how much money is expended on the levee line, it will still be at the mercy of the combination of a muskrat, a dark night, and a careless levee inspector. A burrowing animal can create a hole in the finest levee that has been devised, which, if not closed within a few moments, will insure its destruction. Perfection is difficult in all engineering enterprises. To try to construct a levee line which will resist the river flow under all the imaginary conditions which now fill the public mind, is to attempt the impossible.

RAINFALL CHARACTERISTICS OF THE MISSISSIPPI DRAINAGE BASIN

By H. C. Frankenfield,* Esq.

The Mississippi Basin

The drainage basin of the Mississippi River comprises about all that portion of the United States lying between the Alleghany and the Rocky Mountains except the Great Lakes and Hudson Bay drainages, with a total area of 1 250 900 sq. miles, about two-fifths of the total area of the United States proper. Some water from 31 of the 48 States passes into the Gulf of Mexico. For convenience of study it has become customary to divide the entire area into six major sub-divisions, or sub-basins, as follows: the Missouri, the Upper Mississippi, the Ohio, the Arkansas-White, the Red, and the Lower Mississippi. The drainage areas of the sub-basins, together with their percentages of the entire area, are as given in Table 1.

TABLE 1.—Drainage Area of the Mississippi Basin.

Sub-basin.	Drainage area, in square miles.	Percentage.
Missouri	528 850	42.3
Upper Mississippi	187 850	15.0
Ohio	203 900	16.3
Arkansas-White	186 000	14.9
Red	90 000	7.2
Lower Mississippi	54 300	4.3
Total	1 250 900	100.0

Barometric Pressure Distribution and Precipitation

Having these proportions in mind in a general way, consider the precipitation that occurs over each of these sub-basins, its form to a certain extent, the peculiarities of its distribution, and the reasons therefor, so far as one may be able to give them. Without going deeply into the subject, begin with the ordinary weather chart. Certain relative distributions of barometric pressure will be followed by fair weather, and others by rain or snow. These different distributions have long been grouped into types that officials of the U. S. Weather Bureau have classified according to their regions of inception so far as they have been able to determine them. The types that are principally concerned in precipitation over the Mississippi Drainage Basin are known as the North Pacific, the South Pacific, the Central Rocky Mountain, and the Southwestern. Reference is had now to low-pressure types only, or, technically, cyclones, as these are directly concerned in precipitation causation, although influenced in greater or less degree by the high-pressure types, or anti-cyclones.

Low-pressure areas have warm and moist southeast and south winds on their eastern and southern sides, and the warm and moist masses of air rise until they come into contact with other masses of air which are sufficiently cold to cause condensation of some of the water vapor in the warm masses, and rain

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or snow results. The amount of resulting precipitation will depend upon the velocity and direction of movement of the cyclone and attendant anti-cyclone, the amount of available water vapor, and the difference in temperature between the warm cyclone and the cold anti-cyclone.

The great general and never failing type of rain producer, whatever its origin may be, is a well-defined low-pressure area moving in some easterly direction—usually northeast or east-northeast—with a relatively cold high-pressure area to the northward or northeastward. This is true for the entire Mississippi River Basin, notwithstanding the varying amounts of precipitation that are normal to the different sub-basins. As a matter of fact the four types previously mentioned are neither more nor less than modifications of the great general type, but with different points of origin and different velocities of progression.

Normal Precipitation Over the Mississippi Basin

The figures in Table 2 show the normal precipitation by seasons for the States comprised in the Mississippi Basin. The data are given for the States in each of the six major sub-basins, and wherever a State extends over portions of adjacent sub-basins, the figures cover only that portion of the State within that particular sub-basin. For instance, Kansas would appear in both the Missouri and Arkansas Sub-Basins, etc.

TABLE 2.—Seasonal Normal Precipitation by States, Mississippi Basin.

State.	Drainage area, in square miles.	Precipitation, in inches.				
		September-November.	December-February.	March-May.	June-August.	Total.
Missouri Sub-Basin.						
Montana	121 588	3.00	1.81	4.40	5.75	14.96
Wyoming	69 908	3.15	2.34	4.88	4.24	14.61
Colorado	29 457	3.13	2.02	5.67	6.18	17.00
North Dakota	41 083	2.79	1.48	4.71	7.76	16.74
South Dakota	76 609	3.80	1.79	6.13	8.51	20.23
Minnesota	2 022	5.22	1.49	6.05	10.52	23.28
Nebraska	77 520	4.91	2.09	7.44	10.55	24.99
Iowa	17 289	7.31	3.02	9.13	11.83	31.29
Kansas	42 199	5.96	2.63	7.98	10.57	27.14
Missouri	36 009	8.82	5.17	11.32	12.51	37.82
Canada	15 216	....	....	.....	.....	.....
Entire area	528 850	4.81	2.89	6.77	8.84	22.81
Arkansas-White Sub-Basin.						
Colorado	29 303	3.08	2.51	5.16	6.81	17.56
Kansas	39 959	6.21	3.03	8.28	10.38	27.90
Missouri	14 184	9.68	6.96	13.08	12.81	42.53
New Mexico	17 229	3.49	1.51	4.15	7.65	16.80
Oklahoma	44 057	7.77	4.16	10.36	9.92	32.21
Arkansas	28 013	10.17	10.72	14.52	11.66	47.07
Texas	13 305	4.88	2.15	6.06	7.97	21.06
Entire area	186 000	6.47	4.43	8.80	9.60	29.30

TABLE 2.—(*Continued.*)

State.	Drainage area, in square miles.	Precipitation, in Inches.				
		September-November.	December-February.	March-May.	June-August.	Total.
Upper Mississippi Sub-Basin.						
South Dakota	1 006	4.57	2.43	7.04	9.54	23.58
Minnesota	48 296	5.94	2.28	6.83	11.16	26.21
Wisconsin	38 187	8.10	3.37	8.18	11.61	31.26
Iowa	38 908	7.74	3.59	9.46	11.80	32.59
Illinois	45 448	8.38	6.16	10.48	10.56	35.58
Indiana	2 369	8.34	6.59	10.36	10.21	35.50
Michigan	273	....	....	.....	.....	.....
Missouri	13 363	8.95	6.66	11.32	11.07	38.00
Entire area	187 850	7.43	4.44	9.10	10.85	31.82
Ohio Sub-Basin.						
Illinois	11 413	8.68	8.12	11.36	10.60	38.76
Indiana	30 061	8.93	8.73	11.49	11.00	40.15
Ohio	30 250	8.17	8.86	10.71	11.62	39.36
Pennsylvania	14 680	8.57	9.16	10.50	12.61	41.14
New York	2 136	.....	.....	.....	.....	.....
Maryland	300	8.43	10.99	11.82	13.17	44.41
Virginia	6 902	8.40	10.63	11.26	13.38	43.67
West Virginia	19 567	8.64	11.01	11.75	13.38	44.78
North Carolina	6 581	9.65	13.03	13.60	15.91	52.19
Kentucky	39 398	9.08	11.94	12.38	12.02	45.42
Tennessee	34 124	9.25	13.56	13.96	13.50	50.27
Alabama	6 926	9.16	14.55	14.68	13.15	51.54
Georgia	1 502	....	.....	.....	.....	.....
Entire area	203 900	8.81	10.99	12.14	12.76	44.70
Red Sub-Basin.						
Oklahoma	26 000	7.82	3.97	9.75	9.23	30.77
Arkansas	19 584	9.30	11.37	15.23	10.92	46.82
Texas	28 000	7.47	5.73	10.20	9.32	32.72
Louisiana	16 416	9.28	13.98	14.38	12.07	49.71
Entire area	90 000	8.47	8.76	12.39	10.38	40.00
Lower Mississippi Sub-Basin.						
Missouri	5.914	9.48	10.99	13.12	12.10	45.69
Arkansas	5.738	10.10	12.88	15.31	11.59	49.88
Louisiana	11.300	10.59	14.09	12.93	17.07	54.68
Kentucky	1.200	10.22	13.09	13.40	11.39	48.10
Tennessee	7.898	9.77	13.63	13.92	11.17	48.49
Mississippi	22.250	8.96	15.29	15.93	12.22	52.40
Entire area	54 300	9.85	13.33	14.10	12.59	49.87

Table 2 has virtually been brought down to date (1927), an always desirable condition for those who are interested in the study of normal precipitation, and, furthermore, so far as the writer is aware, the data have not previously been presented in similar form, the usual practice having been to confine the com-

ponent data within the exclusive limits of State boundaries for which river drainage basins have no respect whatever. Table 3 gives the data in Table 2 arranged to show the normal annual precipitation for the sub-basins weighted in accordance with the ratios between their respective drainage areas and the area of the entire basin.

TABLE 3.—Weighted Normal Precipitation, Mississippi Basin.

Sub-basin.	Drainage area, in square miles.	Normal precipitation, in inches.	Percentage.
Missouri	528 850	9.62	30.9
Upper Mississippi	187 850	4.78	15.4
Ohio	203 900	7.29	23.4
Arkansas-White	186 000	4.36	14.0
Red	90 000	2.88	9.3
Lower Mississippi	54 300	2.17	7.0
Total	1 250 900	31.10	100.0

It is readily apparent from even a casual inspection of Tables 2 and 3 that the precipitation over the Mississippi Drainage Basin is at a minimum at its extreme northern and western boundaries, and increases steadily, although by no means uniformly, to the eastward, southeastward, and southward. The meteorological reasons for this distribution are not intricate, and are undoubtedly familiar to the majority of engineers.

Barometric Pressure and Geographical Distribution of Precipitation

As previously stated, low-pressure areas, or cyclones, or storms, as the U. S. Weather Bureau terms them, if they have assumed definite and pronounced formation, as they move eastward over the country, are attended by warm southeast winds on their southeastern and southern sides, or in their southeastern quadrants. These masses of warm air, coming as they do from the Atlantic Ocean, or the Gulf of Mexico, carry with them large quantities of water vapor, and as the air rises higher and higher in the atmosphere, it finally becomes cooled through loss of heat by expansion and by contact with cooler air masses. The ascending air, as it cools, of course loses its capacity for retaining vapor, the saturation point is soon reached, and condensation, or precipitation, sets in. The land nearest the original water supply naturally receives the largest quantity of precipitation, and as the air continues to move farther inland toward the northwest, the available supply of moisture becomes less and less, until the water supply available at the existing temperature has been precipitated, generally at a considerable distance from the Northwestern Rocky Mountains. This is the usual occurrence, although not without exception so far as the surface weather chart shows. At times, there are heavy rains, occasionally torrential, that attend the summer thunderstorms between the Missouri River and the Rocky Mountains. The surface pressure distribution preceding these heavy rains does not always present an orderly arrangement, but aerological observations by kite and balloon will generally show a

mass of cold air, unusually cold for the season, to the westward and northwestward, close to the warm air masses, which will soon under-run them with resulting heavy precipitation.

The Far Northwest then does not receive much moisture from the Atlantic Ocean and the Gulf of Mexico, as the available supply is exhausted farther to the eastward and southward, and its precipitation is due to other types of pressure distribution. One of these is the presence of a warm low-pressure area over Southeastern Wyoming, or Eastern Colorado, with a cold high-pressure area over Saskatchewan, and precipitation, either of rain or snow, will result from the cold northerly and northeasterly winds. This precipitation is usually heavy. Aside from this type precipitation over the Far Northwest generally results from the cold air of the high areas following the movement of low areas across that section. Over the great central valleys the precipitation precedes and accompanies the low area, with little or none following, but in the Far Northwest the southeasterly and southerly winds blowing into the eastward-moving low areas carry but little moisture, not enough to condense until the cold from the following high area reaches it, and precipitation will set in behind the low area and with west and northwest winds. Precipitation from this type of pressure distribution is uniformly small.

Finally, the one great rain-producing type for the entire Central and Southern Mississippi Basin is the "Southwestern." By "Southwestern" is meant a low-pressure area from Mexico, Arizona, or New Mexico (sometimes originating in the Central Pacific Ocean), moving northeastward through Texas and the Ohio Valley, with the usual high area to the northward. The moisture supply is virtually unlimited, and the precipitation is abundant, sometimes in the form of heavy snow in the Ohio and Middle Mississippi Valleys. These storms afford a sufficient explanation of the heavier precipitation over the central and southern portions of the basins and incidentally a partial reason at least for flood behavior in the Lower Ohio and Lower Mississippi Rivers. The rains begin first over these sections, and by the time they reach the Upper Ohio Valley the rivers below have risen decidedly, so that the waters from the great and prolific tributaries of the Ohio and from the upper river itself are added to the already swollen streams below. Sometimes these southwestern storms move in series with intervals of three or four days, and when this happens a severe flood follows. In 1912 there was a series of six of these storms, separated by intervals of only a few days, and the resulting flood was one of the greatest in the history of the Lower Ohio and Lower Mississippi Valleys.

Seasonal Distribution of Precipitation

A study of the seasonal distribution of precipitation over the Mississippi Basin discloses a number of points of difference when the sub-basins are examined separately. In the Missouri and Upper Mississippi Basins the minimum amount occurs during the winter months, December to February. Nearly all the precipitation is in the form of snow which produces comparatively little water when melted. There is a decided increase during the spring

months, March to May, and a still further but less decided increase during the summer months, June to August, a large part of the summer rains coming from thunderstorms. During the autumn months, September to November, there is a sharp decrease as the thunderstorms become less frequent and storm activity is usually at its lowest ebb. In the Ohio Basin there is a steady increase from season to season, beginning with the autumn, although in Illinois and Indiana the winter precipitation is lower on account of the snow effect. The spring and summer data do not differ greatly, the summer thunder showers supplying slightly more rain than the spring storms. In the Arkansas-White Basin the tendency is the same as in the Missouri and Upper Mississippi Basins. The winter decrease from snow effect is not so marked, as it is mostly limited to the western portion of the drainage area, and, furthermore, the rains over the States of Missouri and Arkansas are abundant. In the Red and Lower Mississippi Basins the minimum amount of precipitation is recorded during the autum months, and the maximum during the spring months, the season of the most effective rain-producing storms. The summer months have less rain than the spring months as summer pressure conditions are usually more or less stagnant, and thunderstorms, while frequent, are often local and spring rains are general. Table 4 shows the seasonal amounts for each sub-basin and the percentage of the total fall.

TABLE 4.—NORMAL SEASONAL PRECIPITATION, IN INCHES, AND PERCENTAGES OF NORMAL ANNUAL AMOUNTS, MISSISSIPPI BASIN.

Sub-basin.	September-November.		December-February.		March-May.		June-August.		Total.	
	Amount.	Percentage.	Amount.	Percentage.	Amount.	Percentage.	Amount.	Percentage.	Amount.	Percentage.
Missouri	4.81	21	2.39	11	6.77	29	8.84	39	22.81	100
Upper Mississippi	7.43	23	4.44	14	9.10	29	10.85	34	31.82	100
Ohio	8.81	20	10.99	25	12.14	27	12.76	28	44.70	100
Arkansas-White	6.47	22	4.43	15	8.80	30	9.60	33	29.30	100
Red	8.47	21	8.76	22	12.39	31	10.38	26	40.00	100
Lower Mississippi	9.85	20	13.33	27	14.10	28	12.59	25	49.87	100
Total weighted for areas	6.59	21	5.32	17	9.02	29	10.17	33	31.10	100

SNOWFALL

The correct measurement of snowfall has never proved to be an easy problem. It is true that improved apparatus and methods have resulted in very much greater accuracy during the last thirty years so far as individual places of observation only are involved, but when attempts were made to co-ordinate these individual data over areas of any considerable extent, difficulties arose. Sometimes it was found that the total measured run-off of a stream was greater than the total recorded amount of precipitation of rain

and snow combined, while at other times the percentage of run-off was less than could be reasonably expected. In both instances the trouble was due much more to deficient data than to defective methods of observation. Over broad expanses of comparatively level country this objection could sometimes be waived, but in mountainous regions and in others of diversified topography it is ever present.

For many years it was impossible, except in a few isolated instances, to obtain snowfall data for the high mountain altitudes, although in later years the U. S Weather Bureau, ably assisted by the U. S. Forest Service, has been able to reach some of the high places. Nevertheless, there remains the impossibility of accomplishing satisfactory co-ordination of the data on account of the limited areas covered and the numerous and abrupt changes in topography. If the work of snowfall measurement and its translation into water equivalent is to be properly performed there must be made first a topographical survey of the mountain areas and perhaps some others. Once the survey is made it will stand for all time. Then must come the location of a number of measuring stations that will be sufficiently representative of the entire area; that is, located where they will afford a true index of the snow depths, and entirely without any idea as to uniformity with respect to relative location. The imperative need is for more data properly obtained. During recent years several attempts have been made to secure funds from Congress for a few experimental surveys, but without success. However, there are hopes of the future, especially in view of the fact that the great floods of 1927 appear to have nationalized the water problem, at least to a very large extent. The results of a computation of the normal annual snowfall by the ordinary method of averages are given in Table 5, for the States of the Mississippi Valley, with the small areas of North Carolina, Maryland, Western New York, Michigan, Texas, and New Mexico omitted.

TABLE 5.—Normal Annual Snowfall, Mississippi Basin.

State.	Normal annual snowfall, in inches.	State.	Normal annual snowfall, in inches.	State.	Normal annual snowfall, in inches.
Eastern and Central Montana	55	Missouri	20	West Virginia	34
Eastern Wyoming	59	Kansas	17	Virginia	18
Eastern Colorado	72	Oklahoma	8	Kentucky	16
North Dakota	32	Wisconsin	45	Tennessee	10
South Dakota	36	Illinois	23	Northwest Georgia	4
Nebraska	30	Indiana	26	Northwest Alabama	4
Minnesota	39	Ohio	32	Mississippi	2
Iowa	30	Western Pennsylvania	47	Arkansas	8
				Louisiana	1

As would be expected, the greatest snowfall occurs over the Rocky Mountain districts, with the maximum of about 72 in. in Eastern Colorado. From the mountains eastward and north of the 35th Parallel there is a decrease to the Middle Mississippi Valley, except in Minnesota and Wisconsin, which are partly under the influence of the Great Lakes. Then follows an increase until

the Alleghany Mountains are reached, with a maximum of about 47 in. over Western Pennsylvania. Snowfall below the 35th Parallel is not of consequence.

Roughly, the assemblage of the figures in Table 5 into normals for the individual sub-basins is, as follows:

Basin.	Depth, in inches.
Missouri	39
Upper Mississippi	31
Ohio	26
Arkansas-White	13
Red	6
Lower Mississippi	5

On the usually assumed arbitrary basis of 1 in. of water for each 10 in. of snow, the precipitation of the entire Mississippi Basin, using the data in Table 2, may be divided as shown in Table 6.

TABLE 6.—NORMAL RAIN AND SNOW, MISSISSIPPI BASIN.

Basin.	Rainfall, in inches.	Snowfall (melted), in inches.	Total, in inches.
Missouri	18.91	3.90	22.81
Upper Mississippi	28.72	3.10	31.82
Ohio	42.10	2.60	44.70
Arkansas-White	28.00	1.30	29.30
Red	39.40	0.60	40.00
Lower Mississippi	49.37	0.50	49.87
Entire basin	28.31	2.79	31.10
Weighted	(91%)	(9%)	(100%)

METHODS OF COMPUTING NORMAL PRECIPITATION

The normal values in Table 2 were computed in accordance with the time-honored procedure; that is, by dividing the sum of the normal values at as many units of observation as possible by the total number of units. Little objection could be offered to this plan were it not for the fact that, except in rare instances, the aggregate quantity of data for any considerable area is not sufficient to insure a true and dependable normal. Every hydraulic engineer knows that the amount of precipitation that occurs over so small an area as several hundred square miles is far from uniform, even under virtually identical conditions of topography and meteorological influence. In other words, under the system that has universally prevailed—a system that economic conditions, if nothing more, has rendered compulsory in this country—the data have always been insufficient to serve in the most efficient manner the interests for which they were recorded.

If the number of stations of observation could be multiplied by three or four, or if the stations could be located on the basis of one for not more than each 50 or 75 sq. miles of area, a solution of the problem of questionable normals would be obtained. In mountainous sections, especially in high moun-

tains with their abruptly diversified topography, the stations of observation should be more numerous than one for each 50 or 75 sq. miles, and arranged so as to have proper regard for rapidly changing topography. The Weather Bureau has long realized this situation, but thus far it has been helpless through inability to finance the undertaking. No criticism of other Governmental agencies is implied in this statement, as the Weather Bureau has probably received all the consideration that has been due and in keeping with the requirements of other services of equal importance. Therefore, with a view to the better utilization of the available data in connection with the discussion of flood problems, the Weather Bureau began about 1922 to compile precipitation data by another method which, if laborious, at least possesses the merit of being more accurate, so that the results obtained were very probably as nearly correct as they would have been had the number of stations of observation been multiplied by three or four.

If a planimeter is not available, the plan suggested to the writer by Professor C. F. Marvin, Chief of the Weather Bureau, may be used. It is as follows: Monthly data for a large number of stations were charted and isohyetal lines carefully drawn. These lines were then traced upon sheets of cross-section paper, together with the outlines of the six drainage areas.

The isohyets divide the drainage basins into various small sub-areas, over which the precipitation may be assumed to be uniform and of an amount represented by the mean between the two adjacent isohyetals. Therefore, the number of squares in each sub-area was counted. This number was then multiplied by the average precipitation for the sub-area in question and the product divided by the sum of the counts for all the sub-areas, which latter, of course, is the number of squares in the whole drainage basin being studied. Finally, the sum of the quotients found in the manner described gives the depth of precipitation which, spread uniformly over the whole basin, would represent the same quantity of water as fell in the irregularly distributed precipitation.

This method of discussing precipitation permits the consideration of actual quantities of water over large areas with the certainty that inequalities of rainfall distribution and the uncertainties arising therefrom have been largely eliminated. The method was used in the computation of precipitation data for the great Mississippi floods of 1922 and 1927, and for purposes of comparison the data for the floods of 1882, 1912, and 1913 were computed on the same basis. Monthly seasonal, and unusual normals for the sub-basins and for the different States were also computed, and the sub-basin data are given in Tables 7 and 8.

Analysis of Distribution of Precipitation

General Distribution.—Table 7 shows that the maximum water-fall over the Mississippi Basin occurs in May and June and in equal amount for each month; also, that the minimum fall occurs from November to February, with a maximum difference of only 0.15 in. between any two months.

Considering the sub-basins individually, the same general arrangement is found in the Upper Mississippi, the Missouri, and the Arkansas-White Basins,

those that have the greatest amount of snowfall. The distribution in the Ohio Basin is different in that the maximum fall occurs in March and July and the minimum during October and November, instead of in the winter. In the Red Basin the maximum fall occurs in April and May and the minimum in February, while in the Lower Mississippi Basin the maximum occurs in March, April, and December, and the minimum in October, although, as a whole, there is little variation in monthly values, except during the autumn months.

TABLE 7.—WEIGHTED MONTHLY MEANS OF PRECIPITATION OF THE INDIVIDUAL BASINS OF THE MISSISSIPPI DRAINAGE AREA.

(Precipitation, in Inches.)

Basin.	Percentage of area.	January.	February.	March.	April.	May.	June.	July.	August.	September.	October.	November.	December.	Annual.
Upper Mississippi...	15.0	0.21	0.20	0.30	0.42	0.62	0.63	0.54	0.51	0.51	0.36	0.26	0.20	4.76
Missouri ...	42.3	0.30	0.33	0.49	0.85	1.29	1.36	1.08	0.92	0.84	0.60	0.35	0.32	8.73
Ohio	16.3	0.64	0.58	0.70	0.62	0.64	0.69	0.70	0.63	0.50	0.47	0.47	0.59	7.18
Arkansas-White....	14.9	0.21	0.22	0.32	0.46	0.57	0.50	0.47	0.47	0.39	0.33	0.26	0.24	4.44
Red.....	7.2	0.18	0.17	0.23	0.31	0.32	0.27	0.23	0.23	0.20	0.21	0.19	0.22	2.76
Lower Mississippi...	4.3	0.21	0.19	0.22	0.22	0.19	0.18	0.20	0.18	0.14	0.12	0.15	0.22	2.22
Total.....	100.0	1.75	1.64	2.26	2.88	3.63	3.63	3.22	2.94	2.58	2.09	1.68	1.79	30.09

TABLE 8.—WEIGHTED SEASONAL PRECIPITATION OF THE INDIVIDUAL BASINS OF THE MISSISSIPPI DRAINAGE AREA.

(Precipitation, in Inches.)

Sub-basin.	Ratio to whole basin.	FALL.	WINTER.	SPRING.	SUMMER.	Annual.
		September to November.	December to February.	March to May.	June to August.	
Upper Mississippi.....	0.150	1.13	0.61	1.34	1.68	4.76
Missouri.....	0.423	1.79	0.95	2.63	3.36	8.73
Ohio.....	0.163	1.44	1.76	1.96	2.02	7.18
Arkansas-White.....	0.149	0.98	0.67	1.35	1.44	4.44
Red.....	0.072	0.60	0.57	0.86	0.73	2.76
Lower Mississippi.....	0.043	0.41	0.62	0.63	0.56	2.22
Total	1.000	6.35	5.18	8.77	9.79	30.09

It will be noticed that the Upper Mississippi and Arkansas-White Sub-Basins differ but little as to drainage area and water depth, whereas the Missouri Basin, with nearly three times their drainage area, has less than twice the quantity of water on account of the long snow season. The Ohio Basin,

with a drainage of a little less than two-fifths that of the Missouri Basin, has 82% as much water, the winter season showing an excess over the Missouri Basin. The Red Basin, with about one-half the drainage area of the Arkansas-White and Upper Mississippi, has about 62% and 58%, respectively, of the quantity of water of the two latter basins, while the Lower Mississippi Basin, with about 10% of the drainage area of the Missouri Basin, has about one-fourth as much water, but the Ohio Basin, with a drainage area about two and one-fourth times as large as that of the Red, has nearly three and one-fourth times as much water on account of the smaller precipitation over the Upper Red Basin.

Seasonal Distribution.—The seasonal distribution will conform, of course, closely in character to that previously outlined in the discussion of actual precipitation. A comparison of the annual normals and their percentages of the total basin precipitation by both methods of computation is given in Table 9.

TABLE 9.—Comparison of Precipitation Normals.

Sub-basin.	Actual Precipitation from State Normals.		Computed Precipitation.	
	Inches.	Percentage.	Inches.	Percentage
Missouri	9.62	30.9	8.73	29.0
Upper Mississippi	4.78	15.4	4.76	15.8
Ohio	7.29	23.4	7.18	23.9
Arkansas-White	4.36	14.0	4.44	14.7
Red	2.88	9.3	2.76	9.2
Lower Mississippi	2.17	7.0	2.22	7.4
Total	31.10	100.0	30.09	100.0

The small difference between the two totals is due to the fact that the State normals, after their reduction to the sub-basin normals, were then weighted according to the area ratios. Had the actual precipitation figures been used throughout, the difference would doubtless have been greater, but it was not feasible to undertake the work at this time, and the data were not obtainable from any other source.

Floods

So much for the causes, character, and distribution of precipitation over the great Mississippi Basin. It is in order now to discuss what becomes of this precipitation when at times its normal characteristics undergo decided changes, that is, become abnormal as to quantity, time, and distribution, and manifest themselves in the form of the floods that have become so distressingly and destructively frequent during the last sixteen years, and especially during 1927. The subject of floods will not be discussed herein by the writer, except in connection with the precipitation that caused them.

Before taking up the Lower Mississippi floods, the floods of the Missouri and Upper Mississippi Basins will be touched upon briefly.

Severe or even ordinary floods in the Missouri River above the mouth of the Kansas River are rare. The greatest general flood of which there is record occurred in the spring of 1881 and extended to the mouth of the river, merging there with an Upper Mississippi flood of fair proportions. However, it was not very severe east of Kansas City, Mo. There was also a severe flood in March, 1887, in the North Dakota portion of the Missouri River, but the flood stage was not reached as far south as Omaha, Nebr.

These Upper Missouri floods are spring floods, and are due usually to a combination of melting snows in upper reaches, high temperatures, and rain. The melted snow factor is not of importance unless in large quantity, and supplemented by substantial rains, and the rain is the essential factor. It is not necessary to discuss the floods caused by the formation and subsequent breaking of ice gorges, and, incidentally, it is very probable that the flood of 1881 was largely an ice flood. From Kansas City eastward, Missouri River floods are quite frequent, although as a rule not great. They are essentially rain floods and occur usually in late June and in July. By far the greatest of these floods occurred in June, 1844, a season of torrential rainfall that also extended over the Middle Mississippi Valley causing stages that have not since been reached between the mouth of the Illinois River and Cape Girardeau, Mo.

Upper Mississippi Floods, that is, floods above the mouth of the Missouri River, are more numerous. They are almost entirely spring floods, and, unless caused by ice gorges, they are rain floods, although the rain may be considerably augmented at times by melted snows. Owing to the flat contour, these floods are likely to be destructive below the mouth of the Wisconsin River.

The floods of the Ohio, Arkansas-White, Red, and Lower Mississippi Valleys are virtually of annual occurrence, and great floods are not infrequent, as witness those of comparatively recent occurrence, 1912, 1913, 1915, 1916, 1920, 1922, and 1927. These floods are pre-eminently rain floods, although at times considerable assistance is lent by melting snows in the winter or early spring from the Ohio River and its many large mountain tributaries.

The Flood of 1927

As the flood of 1927 overshadowed all others from Cairo, Ill., southward it will hardly be necessary to discuss the precipitation conditions for any others except the flood of 1922 which was next in magnitude to that of 1927. All data will be expressed in terms of water depth over the entire area.

In Table 10 will be found the data for the floods of 1922 and 1927, together with the departures from the normals. The normals are of recent determination and are the product of the combined labors of Messrs. Montrose W. Hayes and Walter J. Moxom, of the U. S. Weather Bureau Office, at St. Louis, Mo., and engineers of the office of the Mississippi River Commission, also of St. Louis.

The total depth of water more or less contributory to the flood of 1927 was 10.79 in., and 12.38 in. if the last two weeks of December, 1926, are included, as they really should be. The excess of 1927 over 1922 for the entire period, December 18 to April 30, was 1.80 in., but for the four months, January to April, 1927, only 0.21 in., or a little less than 2 per cent. The excesses

above the normal amounts in both years are significant. The water depths for January were below the normal amounts in both 1927 and 1922, and February, 1927, was also deficient, while February, 1922, shows an excess. Taken together, January and February, 1927, show a deficiency of 0.22 in. against a deficiency of 0.15 in. in 1922, a difference so small that during the two months both years were virtually on even terms. The month of March shows a decided excess for both years, but with 1922 still in the ascendancy, the figures being + 0.72 in. for 1927 and + 1.24 in. for 1922. Both values are quite large and indicate a severe flood. In April, the month of greatest water depth over the entire area, conditions became decidedly reversed with an excess of 1.75 in. in 1927 against 0.94 in. in 1922.

TABLE 10.—PRECIPITATION FOR THE FLOODS OF 1922 AND 1927, IN TERMS OF INCHES OF WATER OVER THE ENTIRE MISSISSIPPI DRAINAGE AREA, AND DEPARTURES FROM NORMAL.

(Absence of sign indicates plus departure.)

Sub-area.	Drainage, in square miles.	Flood of 1922.									
		January.		February.		March.		April.		Total.	
		Amount.	Departure.	Amount.	Departure.	Amount.	Departure.	Amount.	Departure.	Amount.	Departure.
Upper Mississippi	187 850	0.16	—0.05	0.31	0.11	0.35	0.05	0.53	0.11	1.35	0.22
Missouri	528 850	0.29	—0.01	0.44	0.11	0.89	0.39	1.48	0.63	3.10	1.12
Ohio	203 900	0.44	—0.21	0.41	—0.12	0.92	0.21	0.66	0.04	2.43	—0.08
Arkansas-White	186 000	0.18	—0.03	0.31	—0.01	0.60	0.28	0.62	0.17	1.61	0.41
Red	90 000	0.19	0.01	0.22	0.05	0.40	0.17	0.36	0.05	1.17	0.28
Lower Mississippi	54 300	0.19	—0.02	0.21	0.02	0.36	0.14	0.16	—0.06	0.92	0.08
Total	1 250 900	1.45	—0.31	1.80	0.16	3.52	1.24	3.81	0.94	10.58	2.03

Sub-area.	Flood of 1927.										December 18–31, 1926.		1927 total, including December 18–31, 1926.	
	January.		February.		March.		April.		Total.					
	Amount.	Departure.	Amount.	Departure.	Amount.	Departure.	Amount.	Departure.	Amount.	Departure.	Amount.	Departure.	Amount.	Departure.
Upper Mississippi	0.15	—0.06	0.15	—0.05	0.40	0.10	0.64	0.22	1.34	0.21	0.08	—0.01	1.42	0.20
Missouri	0.25	—0.05	0.28	—0.05	0.63	0.14	1.59	0.74	2.75	0.78	0.07	—0.07	2.82	0.71
Ohio	0.67	0.02	0.54	0.01	0.82	0.11	0.95	0.33	2.98	0.47	0.70	0.43	3.68	0.90
Arkansas-White	0.30	0.09	0.16	—0.06	0.48	0.16	0.69	0.24	1.63	0.43	0.22	—0.11	1.85	0.32
Red	0.17	—0.01	0.17	0.00	0.29	0.06	0.42	0.11	1.05	0.16	0.25	—0.15	1.30	0.01
Lower Mississippi	0.16	—0.05	0.18	—0.01	0.37	0.15	0.33	0.11	1.04	0.20	0.27	—0.17	1.31	0.03
Total	1.70	—0.06	1.48	—0.16	2.99	0.72	4.62	1.75	10.79	2.25	1.59	—0.08	12.38	2.17

Comparison of the data as a whole for 1922 and 1927 does not disclose any particular reason why the magnitude of the flood of 1927 should have exceeded

that of 1922. In fact, the behavior was much the same in both. The months of January and February differed but little, and, taken together, both were deficient. In March, 1922, the excess precipitation was large and the maximum flood occurred in that month, while in April, 1927, the excess was nearly twice as great as in April, 1922, and the maximum and greater flood occurred in that month. Therefore, it is obvious that the distribution of precipitation, as to intensity, time, and locality, is almost as important as the amount thereof. In both 1922 and 1927 a reasonable additional amount in January and February and an equivalent lesser amount in March and April would have lowered the flood crests materially, while the reverse occurrence would have resulted in still higher crests, assuming that levees had remained intact. In 1912, another year of great March and April flood, the excess water in February and March amounted to 1.57 in., with a slight deficiency in January, and the total quantity of water for the three months was 8.24 in., 2.34 in. less than in 1922, and 2.55 in. less than in 1927. The 1912 flood was caused by a series of six storms of the "southwestern" type, the best rain producers, as stated previously, for the Middle and Lower Mississippi Basins. These six storms were separated by intervals of only a few days, a period entirely too short to check the rate of rise materially, and another inch or so of water from the storms would probably have resulted in a flood as great as that of 1922 had the levees remained intact. It should be stated, also, that during the flood of 1912 neither the Upper Mississippi, Arkansas, nor Red Rivers was in severe flood.

In the floods of 1882, 1903, and 1913, the total quantities of water, January to March, inclusive, were 7.68, 6.11, and 6.44 in., respectively. These figures appear to justify the assumption, partly at least, that a water cover over the Mississippi Basin of about 10 in. in 3 or 4 months between January and April, will probably result in a great flood from Cairo southward, and that under the most favorable conditions for flood formation, a somewhat less amount will answer.

All data thus far given have been based upon precipitation and water depth over the entire Mississippi Basin, although it is a recognized fact that the Missouri River above the mouth of the Platte, and the Mississippi River above the mouth of the Wisconsin do not contribute materially to Lower Mississippi floods. If these upper areas, amounting to approximately one-third of the total drainage, were to be excluded, the precipitation over the remaining two-thirds would have amounted to 16.28 in. of water over the latter area in 1927, whereas in 1922 the total water depth was 13.70 in., 2.58 in. less than in 1927. If the data for the last two weeks in December, 1921, and 1926, were included, the 1927 excess would be still larger.

Possible Maximum Flood Crests in 1927

A question frequently asked the U. S. Engineer Corps and the Weather Bureau was this: What would have been the maximum crests reached during the flood of 1927 had all the main levees remained unbroken? Each organization considered the question independently and by somewhat different methods, and opportunity was afforded to make informal comparison of the Weather Bureau figures with those of the Engineer Corps in Washington.

It is gratifying to be able to state that differences in the two groups of figures were insignificant. The stages reached at key stations and the estimated possible stages under most favorable conditions are given in Table 11.

TABLE 11.—ACTUAL AND POSSIBLE CREST STAGES, FLOOD OF 1927.

Station.	River.	Flood stage, in feet.	Crest, 1927, in feet.	Possible stage, 1927, in feet.	Difference, in feet.
St. Louis, Mo.	Mississippi	30	36.1	36.1	0.00
Paducah, Ky.	Ohio	43	47.2	48.0	+0.08
Cairo, Ill.	Ohio	45	56.4	57.7-58.0	+1.3—+1.6
Memphis, Tenn.	Mississippi	35	46.0	47.2-47.5	+1.2—+1.5
Helena, Ark.	Mississippi	44	56.8	58.2-58.5	+1.4—+1.7
Arkansas City, Ark.	Mississippi	48	60.5	68.5-69.0	+8.0—8.5
Greenville, Miss.	Mississippi	42	54.7	61.5-62.0	+6.8—7.3
Vicksburg, Miss.	Mississippi	45	58.7	64.5-65.0	+5.8—6.3
Natchez, Miss.	Mississippi	46	56.5	64.5-65.0	+8.0—8.5
Baton Rouge, La.	Mississippi	35	47.8	54.5-55.0	+6.7—7.2
New Orleans, La.	Mississippi	17	21.0	27.2-27.7	+6.2—6.7

The processes of reasoning through which the figures in Table 11 were evolved were basically alike, and perhaps a single example will suffice to illustrate them.

At Cairo the crest stage of 56.4 ft. occurred on April 20. The crevasse at Dorena, Mo., on the right bank, 30 miles below Cairo, occurred at 4:00 A. M., April 16, and after that time the river at Cairo rose only 0.7 ft., notwithstanding the fact that the Mississippi River at St. Louis was at a stage of 34 ft., or 4 ft. above the flood stage, and continued to rise steadily for a week thereafter. The Ohio River, at Paducah, Ky., also continued to rise for a few days after the crevasse. The rises at St. Louis and Paducah after the Dorena crevasse were about 2.0 and 0.9 ft., respectively, with an increase of only 0.7 ft. on the Cairo gauge. It is apparent, therefore, that if the crevasse had not occurred at Dorena, and making the usual allowance for the additional rise that would ordinarily have resulted from the high stage at St. Louis, the crest stage at Cairo would have been approximately 58 ft. about, but not after, the end of April. Incidentally, with the stages at Cairo and St. Louis as they were on the morning of April 16, and no crevasse at Dorena, but with the Upper Ohio in flood equal to that of March, 1913, the crest stage at Cairo would probably have approximated 62 ft.

Finally, it is within the bounds of possibility that the indicated stages in Table 11 could be exceeded. It can be shown, for instance, that stages of 65 ft. at Cairo, and of 73 ft. at Arkansas City, are not entirely removed beyond the limits of possibility under the most favorable conditions of precipitation distribution and synchronization. However, the probability is so very remote that its complete realization can perhaps be disregarded, as the sequences of the proper meteorological combinations that would be necessary to produce the extreme conditions as computed, while not absolutely impossible, are almost beyond belief.

RUN-OFF CHARACTERISTICS OF THE MISSISSIPPI RIVER DRAINAGE BASIN

By Nathan C. Grover,* M. Am. Soc. C. E.

Flood problems in general can be presented most satisfactorily to engineers and the interested public in the simple terms of quantities of water and capacities of channel. In such simple terms the flood problems of the Mississippi may be said to consist in determining the quantities of water to be expected in various stretches of the river and in making ample provision in channel and temporary storage capacities for conveying those quantities to the Gulf. If these problems are to be thus set forth, it is necessary that reliable continuous records of discharge be collected for the critical stretches of the Mississippi River below Cairo, Ill., and for its major tributaries. Unfortunately, such records, covering the crests of floods and showing the possibilities of their synchronization, have not been made, and this basic information is, therefore, not now available.

A few approximate figures may serve to make the situation clearer. A typical flood at Cairo, consisting of the combined discharges of the Ohio River and of the Mississippi River above the Ohio, is 2 000 000 cu. ft. per sec., more or less. The tributaries below Cairo may add to such a flood enormous quantities of water, amounting in 1927 to more than 1 500 000 cu. ft. per sec., according to the best estimates that have been available to the writer, making an apparent flood discharge of more than 3 500 000 cu. ft. per sec. for which channel and storage capacities must be provided so far as they do not already exist. The maximum capacity of the channel at New Orleans, La., with the levees intact is about 1 500 000 cu. ft. per sec. Any satisfactory plan of flood control must afford the means for handling the water in the lower river in excess of the channel capacity at New Orleans by providing either additional channels to the Gulf, definitely set aside for that purpose, or sufficient lateral storage capacity to detain the excess water until it can be discharged through present recognized channels, or, more probably, channel capacities to carry the major part of the floods and supplementary lateral storage to take care of the remainder.

The writer does not wish to be understood as stating that the discharge of the Mississippi in 1927 reached in any stretch at any time the enormous total of 3 500 000 cu. ft. per sec.; some channel and lateral storage had been left available to the river, many crevasses were made whereby the river re-occupied its natural lateral basins that had been closed by levees, and there was a great quantity of water that broke through the artificial barriers that had been constructed, and re-occupied the natural channels across the delta to the Gulf, all of which, by returning in part to Nature's plan, tended to decrease the crest discharges in the lower river, which have been aggravated

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by the very works designed to control them. In spite of these mitigating conditions, however, the waters in this flood covered nearly 20 000 sq. miles, situated in seven States, an area approximately equal to that of New Hampshire and Vermont combined.

A comparison of the Mississippi with other great rivers of the Continent will serve to emphasize the magnitude of the quantities of water to be controlled in a Mississippi flood. Available estimates indicate that the total run-off in a great flood of the Mississippi, covering a period of six months or more, may exceed 500 000 000 acre-ft.—a quantity equal to the average annual run-off of that river or to the total run-off of either the Columbia or the St. Lawrence for three years, or of the Colorado for more than thirty years. The seven largest reservoirs constructed by the U. S. Bureau of Reclamation in the Western States have an aggregate capacity of 7 280 000 acre-ft., or about 1½% of the estimated 500 000 000 acre-ft. in a single Mississippi River flood. The proposed Boulder Canyon Dam, with a capacity of about 25 000 000 acre-ft., would store the total run-off of the Colorado for nearly 1½ years, and yet this capacity, great as it is, is equivalent to only 5% of the run-off in a single flood of the Mississippi.

The area from which the water of the Mississippi is derived, aggregating 1 240 000 sq. miles, is widely diversified in topography, geology, vegetation, and climate. It constitutes the great central basin of the country, extending from the crest of the Alleghanies to the crest of the Rockies and from the Great Lakes to the Gulf of Mexico, and includes mountains and plains, forests and grasslands, fields that are cropped annually, and remnants of the great public range. This great area includes all or parts of 31 States and of 2 Provinces of Canada, and is equivalent to nearly 41% of the area of the Continental United States. It is pre-eminently a great food-producing region and contains about 42% of the population and 39% of the wealth of the country. No other river basin on the globe combines topography, soil, and climate to make attractive living conditions for so many people as that of the Mississippi. It has within its borders many cities, railroads, highways, and farms with artificial or modified drainage conditions. These works of Man, which have certainly affected the rate and probably the total quantity of discharge of the Mississippi, cannot be decreased, however desirable it might be in the interest of flood control to restore the basin to the conditions of Nature. In fact, to restore it to such conditions would eliminate the need for flood control, because no one would be there to be damaged. Rather, it is certain that such works will be continually increased with the growth in population, industry, transportation, and wealth of the country. It may be possible, however, to improve present conditions by a reasonable program of forestation and by the construction of reservoirs in head-water streams, but, in the writer's opinion, it is vain to hope for a material reduction of the flood flows of the Lower Mississippi by such methods. Although it is recognized that vegetation, including forests, affects run-off and that reservoirs on head-water tributaries may be operated so as to decrease in a small way the floods in the lower

river, the quantities of water in a Mississippi flood are so great that these two influences, which will always be considered beneficent, can probably never have a major effect and at most can only add some small amount to a reasonable factor of safety which should be contained in any plan of flood control.

The Mississippi above the mouth of the Missouri and the four principal tributaries—the Missouri, Ohio, Arkansas, and Red—drain 96% of the total area of the Mississippi Basin. There are gauging stations which will yield continuous and reliable records at all stages of discharge from about 86% of the area drained by rivers above Cairo. Below Cairo, however, the run-off from only 18% is fully measured, and this 18% does not include any appreciable part of the valley that is subject to torrential rains.

There are several long-time records of the stage of the Mississippi between Cairo and the Gulf, and many measurements of discharge have been made by current meters and floats whereby some of the gauges have been fairly well rated for conditions of unbreached levees. There are, however, in this part of the basin no records of stage that can be satisfactorily converted to records of discharge for the varying conditions of slope and of flow through lateral flood channels resulting from crevasses. In other words, there are no reliable records of crest discharge of either the main river, or of its tributaries below Cairo, and the existing voluminous records are scattered through hundreds of reports, in part out of print, and are too lacking in adequate analysis, interpretation, and co-ordination to be reasonably usable.

The eastern and northern tributaries drain regions of moderate precipitation that accumulates in part during the winter in the form of snow and ice, which thus form the source of the characteristic spring freshets that occur when the rising temperatures release the water. The head-water feeders of the southern, western, and northwestern tributaries drain large regions of semi-aridity, in which the precipitation decreases gradually toward the west. More than one-third of the basin of the Mississippi lies west of the 100th Meridian which passes through the middle of the Dakotas, Nebraska, and Kansas. This part yields considerably less than 5% of the water of any flood. By contrast, the Ohio, draining 17% of the basin, may yield 50% or more of a Mississippi flood, and the lower basins of the southern tributaries and the lower valley of the Mississippi itself, which are subject to heavy rains that may occur in any month with the intensity of cloudbursts, yielding several inches of rain in 24 hours, may furnish a considerable and critical part of the flood waters.

An illustration of the differences in run-off between the humid and dry parts of the western and southern tributaries is afforded by the recorded run-off of the Missouri River for the year October 1, 1925, to September 30, 1926, at Leavenworth, Kans., and at Boonville, Mo. At Leavenworth, with a drainage area of 428 000 sq. miles, the run-off was 26 400 000 acre-ft.; at Boonville, with a drainage area of 508 000 sq. miles, or only 20% more than that at Leavenworth, the run-off was 40 900 000 acre-ft., or nearly 55% more.

The foundation or base of every great flood in the Mississippi comes from the Ohio. Starting with at least a reasonably large flood in the Ohio, the

building up of a great Mississippi flood depends on the synchronizing of that flood with floods from other tributaries as the crest progresses toward the Gulf. The peaks of the Ohio floods generally pass into the Mississippi between January 1 and May 31. They frequently join with high water from the Upper Mississippi at Cairo, as the records of stages at St. Louis show that April, May, June, and July are the months in which crest stages generally occur at that place. Fortunately, however, for floods at Cairo, the crest stages from the Ohio are generally ahead of the crest stages from the Upper Mississippi, but as crest stages may occur in either river in April and May, there is a possibility that in some years the crests of an early flood from the Upper Mississippi and a late flood from the Ohio may reach Cairo simultaneously. Nevertheless, it is probable that the climatic conditions that cause early floods on the Upper Mississippi also cause them on the Ohio—in other words, that climatic conditions make the possibility of a combination of crests of floods from the two rivers at Cairo rather remote.

If there were no influx of flood waters below Cairo, the flood problems of the Lower Mississippi would probably be relatively simple, because the natural channel and flood-plain storage would so flatten the crests of the floods passing Cairo as to permit the water to pass without causing damage through those stretches of channel in the lower river that have smaller capacities than the channel just below the mouth of the Ohio.

The yield of the lower tributary basins and especially of the parts of those basins that lie within 200 miles on either side of the Mississippi, in the path of the big storms, is, therefore, of great importance in any study of floods, and it is just this part of the run-off that is largely unknown. The contribution of the lower part of the basin to the disastrous floods of the Mississippi is caused exclusively by heavy rains that occur most frequently from October to March, but that come occasionally in the late spring and early summer. The rate of local run-off is dependent on the magnitude and intensity of the precipitation over this southern part of the basin. The probability of floods in the Ohio synchronizing with floods in the lower tributaries of the Mississippi appears, therefore, to be much greater than that of floods in the Ohio synchronizing with floods in the Upper Mississippi.

The floods of the Mississippi are thus caused in the first instance by floods in the Ohio to which melting snows in the head-water tributaries contribute, but with which must be combined high water from the Mississippi above Cairo or floods in the southern tributaries due to heavy rains over broad areas of the lower basin, or by some combination of these conditions. The combination of climatic conditions that produces floods has occurred periodically for countless ages and will be repeated again and again as years go by, and floods will surely recur with those conditions. De Soto experienced a great flood when he visited the Mississippi, but it caused no great damage because there was little to be damaged. Future floods may be no greater than that of De Soto's time, but if they are uncontrolled they will be increasingly destructive in lives and property as the use of the great flood-plains of the delta increases.

The flood-plains of the Mississippi and its delta have been built by its floods. The lands that are now protected by levees from overflow owe their very existence to floods. The process of valley and delta building involves periodic inundation and the deposition of silt. River channels through the flood-plains and delta do not normally have sufficient capacities to carry within their banks the floods that occur. A change from this fundamental condition of overtopped banks and temporary lateral storage of water is, therefore, man-made and represents a radical departure from the natural processes of a silt-bearing river that is constantly building its delta. It must be recognized that such an artificial change in the regimen of a great river is always attended by difficulties and dangers and that to be adequate, it must be on a scale commensurate with that of the natural forces.

In Nature's process of flood-plain building, a Mississippi flood, after leaving Cairo, would normally spread out over several thousand square miles of bottomland included in the St. Francis, Tensas, Yazoo, and other basins, and would be drained therefrom over a period of several months at a fairly uniform but lesser rate than the maximum inflow to those basins. Consequently, the channels leading from the basins to the Gulf would be formed with smaller capacities than the channels discharging water into the basins. So long as the great overflow basins were available for temporary storage of excess flood waters, the smaller channel capacities across the lower delta were ample. As Man has gradually shut off these natural regulating basins by building levees, thereby depriving the river of their effects in equalizing flow, the floods below the basins have inevitably increased in magnitude, and greater channel capacities have been required. Available records of stage show clearly this situation, which should have been expected as surely as increased fluctuations in the stage and discharge of the St. Lawrence would follow if river channels were to be constructed through the Great Lakes so as to eliminate the equalizing effects afforded by the natural storage of those wonderful natural basins.

The run-off characteristics of the Mississippi Basin above Cairo are fairly well known and are not being changed radically by man, but those below Cairo have been and are being gradually changed with progress in the construction of levees, and there are no satisfactory records of the resulting changes in crest rates of discharge. The run-off characteristics will continue, of course, to change until stability has been reached in a system of levees, spillways, and overflow basins that will carry uniformly the successive floods of the river.

Students of hydrology have long realized that a flood like that of 1927 was inevitable. They know, too, that similar and even larger floods will occur in the future. To the end that the magnitude and periodicity of these certain great floods may be known within reasonable limits of error, it is necessary that the yield of water by the tributaries shall be definitely known over a period of years, not only in total quantities, but in distribution throughout the year and in variations from year to year. On no other basis can satisfactory forecasts of future floods be made as a necessary first step in designing control works.

The re-occupation of leveed basins and the breaking of barriers to natural overflow channels to the Gulf cannot, of course, be longer left to chance, but such capacities of channels and overflow basins must be provided in advance of flood as to remove the danger of disastrous breaks in levees. It may be stated, as a self-evident truth, that the nearer the methods of flood control conform to Nature's way the less will be the difficulty in maintaining the control works, and, conversely, the greater the departure from Nature's way the stronger will be the tendency for the river to return to natural conditions and the more difficult will be the maintenance of these works.

It is the writer's opinion that an adequate determination of the necessary channel and storage capacities can be made only on the basis of reliable estimates of the quantities of water for which provision must be made in the several stretches of the river, and such estimates must rest on continuous reliable records of discharge. Similarly, the problem of selecting the best locations for temporary storage of the flood waters in lateral basins and the most practicable auxiliary channels to the Gulf cannot be intelligently studied or correctly solved until the area subject to floods has been topographically mapped. As less than 8% of this area is now mapped, the problem is, indeed, difficult.

It is fortunate that the same records and maps will serve as a basis for both flood control and general development. The problems arising in connection with the utilization and administration of surface waters in the Mississippi Valley are constantly increasing. The best solution of many of these problems can be reached only on the basis of adequate data in regard to quantities of run-off and its distribution with respect to both time and drainage areas. As utilization progresses, these data are becoming more essential and are increasingly in demand by a wide range of users, including Federal, State, and municipal officials, corporations, and individuals. Definite plans for flood-control works and for important structures to utilize rivers, or to be built on their banks or flood-plains, should not, in the interest of safety and economy, be adopted in the absence of adequate hydraulic investigations and hydrologic data, or of complete topographic maps.

The many failures of enterprises developed on the basis of insufficient records of water supply show conclusively that it is unsafe to proceed with developments involving the use of rivers except on the assurance afforded by adequate records of discharge. That estimates of water supply made from records of rainfall and comparisons of drainage areas may be unreliable and misleading has been demonstrated repeatedly, and the Mississippi Valley is no exception to the general rule. The wise development of this valley must rest on adequate data in regard to the discharge of its many rivers. Because of the diversity of interests, of which flood control is doubtless the most pressing, and the complexity and interrelation of the many problems involved, provision should be made for the collection of these necessary data on a continuing basis, and for their analysis, correlation, and publication promptly, in order that they may be available for use by engineers and the public.

The floods of the Mississippi affect the development and prosperity of a region that is of major importance, not only to the United States, but to the world. As population increases and development progresses, the resources of this vast region must ultimately be completely utilized in the production of foodstuffs and textile raw materials needed by the world. Whatever effects the levees and other works for controlling floods may have on the run-off characteristics of any part of the basin, the control must be accomplished in the interest, not only of the flooded regions, or the entire Mississippi Basin, but of the whole country. Full justice cannot be done if the situation is approached with a narrow-angle view and near-sighted vision.

THE WORK OF THE MISSISSIPPI RIVER COMMISSION

By C. W. Kutz,* M. Am. Soc. C. E.

The Mississippi River Commission was created by an Act of Congress approved June 28, 1879. It consists of seven Commissioners, three of whom are selected from the Corps of Engineers, U. S. Army; one from the U. S. Coast and Geodetic Survey; and three from civil life, two of whom must be civil engineers.

The basic law required the Commission to "mature a plan or plans as will correct, permanently locate and deepen the channel, and protect the banks of the Mississippi River; improve and give safety and care to the navigation thereof; prevent destructive floods; promote and facilitate commerce, trade and the postal service."

It also called for a report in full on the practicability, feasibility, and probable cost of the various plans known as the jetty system, the levee system, and the outlet system, as well as on such others as the Commission deemed necessary. Before presenting the conclusions of the Commission as embodied in its first report, it seems desirable to describe briefly the physical characteristics of the Lower Mississippi Valley and the work that had been done prior to the creation of the Commission and, particularly, to refer to the prior Federal investigations and surveys, as on the results of those studies the plan of the Commission was largely based.

Physical Characteristics

The Mississippi River and its tributaries drain an area of 1 240 000 sq. miles, including all or portions of 31 States and about 20 000 sq. miles of Canada.

The natural divisions of the basin are as shown in Table 12.

TABLE 12.—Divisions of the Mississippi Basin.

Designation.	Area, in square miles.	Ratio to entire basin.
Upper Mississippi Basin	165 900	0.13
Missouri Basin	527 100	0.43
Ohio Basin	201 700	0.16
Arkansas Basin	186 300	0.15
Red Basin	90 000	0.07
Central Valley	69 000	0.06
Total	1 240 000	1.00

The alluvial valley, with which the Commission is chiefly concerned, begins at Cape Girardeau, Mo., 52 miles above Cairo, Ill. This flood-plain of alluvial deposit varies in width from 20 miles just north of Natchez to 80

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miles at Greenville, Miss., the average width above Plaquemine, La., being 45 miles. Below Plaquemine it spreads out into a broad delta, the general shape of which is a semi-circle having a radius of about 80 miles.

Through this valley the river winds its way to the Gulf, generally following the bluffs on the east from the mouth of the Ohio to Memphis, Tenn., touching the west bluffs at Helena, Ark., and hugging the east bluffs closely from Vicksburg, Miss., to Baton Rouge, La. The air-line distance from Cape Girardeau to Head of the Passes is about 600 miles; the river distance, 1 102 miles.

The river flows in a well-defined but unstable channel, the natural banks of which are from 3 to 12 ft. above the general level of the plain. These banks are highest nearest the river and have an average slope away from the river of 7 ft. per mile in the first mile. The river is constantly eroding its banks in unprotected bends and forming new land on points. This caving is accentuated in many places by an underlying strata of sand which washes out and causes the bank to fall by its own weight as the river subsides.

Above the mouth of Red River on the right bank, and above Baton Rouge on the left bank, the drainage of the valley finds its way into the river through tributaries. Below these points the drainage is carried away from the river and into the Gulf of Mexico through numerous bayous and streams.

The minimum measured discharge of the river at Columbus, Ky., below the mouth of the Ohio, is 71 000 sec-ft.—the maximum at this point, 2 015 000 sec-ft. having occurred in 1912. The carrying capacity of the natural channel at bank-full stage is about 1 000 000 sec-ft.

The building of levees on the banks of the Mississippi River began with the first settlements in the lower alluvial valley (as early as 1717), and as a measure of economy they were built on the highest ground, that is, near the river bank. The extension of levees kept pace with the establishment and growth of settlements along the lower river, each planter being required to complete the levee along his own river-front. In 1812, when Louisiana was admitted to the Union, the levees extended on both sides of the river, to Baton Rouge on the left bank and to Point Coupee on the right bank. By 1844, they were almost continuous from 20 miles below New Orleans to the mouth of the Arkansas River on the right bank and to Baton Rouge on the left bank.

The floods of 1849 and 1850 caused widespread damage in the valley. National interest was aroused and, in response to a plea for Federal aid, the Swamp Land Act was passed in 1850. By this Act the Federal Government granted to the several States in the Lower Mississippi Valley all unsold swamp lands and overflowed lands within their limits to provide funds to reclaim the districts subject to overflow. The States of Louisiana, Mississippi, Arkansas, and Missouri organized offices for the sale of swamp lands and appointed Commissioners for the location and construction of levees. Due, however, to differences in laws governing levee construction, to differences between counties and parishes within a State, and to a total lack of co-ordination and co-operation, but little effective protection was secured from the proceeds of the sale.

In the same year, Congress appropriated $50 000 "for a topographical and hydrographical survey of the delta of the Mississippi with such investigations as may lead to determine the most practical plan for securing it from inundation * * *". This work was placed in charge of Capt. A. A. Humphreys, Corps of Engineers, U. S. A. Field work was started in 1851, but suspended in the fall of that year until 1857 when it was resumed by Lieut. Henry L. Abbot, Corps of Engineers, U. S. A., who had been detailed as Assistant to Captain Humphreys. The final report on this survey was submitted jointly by these officers in 1861.*

This report, which is very comprehensive, discusses, as plans for protection, a system of cut-offs, the diversion of tributaries, a system of head-water reservoirs, and a system of outlets as alternatives for levees. Its conclusions were that no advantage could be derived from diverting tributaries or constructing reservoirs, that the plan of cut-offs and new or enlarged outlets to the Gulf were too costly and too dangerous to be attempted, and that levees could be relied upon for protecting all the alluvial bottom-lands liable to inundation below Cape Girardeau. It noted, however, the possibility of reducing levee heights near Lake Providence, La., by constructing near that town an outlet leading to Bayou Tensas and the Black River, but its approval of this alternative was conditional upon further investigation.

The Humphreys and Abbot investigation included measurements for the determination of the quantity of sediment in suspension in Mississippi River water. These observations were begun at Carrollton, La., in 1851, and continued for two years, and the results, combined with similar measurements made by other agencies, led to the following conclusions:

(*a*) "* * * the Mississippi water is not charged to its maximum capacity with sediment; * * * A most important practical deduction may be drawn from this fact, namely, the error of the popular idea that a slight artificial retardation of the current, that caused by a crevasse, for instance, must produce a deposit in the channel of the river below it.

(*b*) "* * * the sediment of the Mississippi is to the water, by weight, nearly as 1 to 1 500, and by bulk, nearly as 1 to 2 900; provided long periods of time be considered. * * * and if the mean annual discharge of the Mississippi proper be correctly assumed at 19 500 000 000 000 cubic feet, it follows that * * * sedimentary matter, constituting one square mile of deposit 241 feet in depth are yearly transported in a state of suspension into the gulf.

(*c*) "Besides the amount held in suspension, the Mississippi pushes along into the gulf large quantities of earthy matter.

"No exact measurement of the amount of the annual contributions to the gulf from this source can be made, but from the yearly rate of progress of the bars into the gulf * * * it appears to be * * * which would cover a square mile about 27 feet deep.

"The total yearly contributions from the river to the gulf amount, then, to a prism 268 feet in height, with a base of one square mile. * * *"

The next milestone in the history of the Lower Mississippi was an Act of Congress (1874) appointing a Joint Civil and Military Commission to make a full report upon the best system of permanent reclamation and redemption of the alluvial valley from inundation. The report of this Board, submitted in

* "Report upon the Physics and Hydraulics of the Mississippi River," 1861.

1875, was based upon the results of the Humphreys and Abbot survey, and upon additional data secured subsequent thereto. Its conclusions were: "No practicable aid can be derived from any diversion of tributaries or by artificial reservoirs; that cut-offs are very pernicious and that outlets, though correct in theory, find no useful application to the Mississippi". A general system of levees was recommended.

The Mississippi River Commission was created four years later. In its first report to the Secretary of War (1880), it advocated as the best means of securing the needed improvement in its navigation, a plan which would concentrate, rather than disperse, the waters of the river. In this report it also condemned the outlet system as a means of flood control, expressing the belief that "no surer method of ultimately raising the flood surface of the river can be adopted than by making lateral outlets for escape of its flood waters."

Asserting that the bad navigation of the river was produced by the caving and erosion of its banks, the excessive widths, and the bars and shoals resulting directly therefrom, it proposed to narrow the river, where too wide, by contraction works and to protect the caving banks by bank revetment. This was assumed by the Commission to be the plan referred to in the Act as the "Jetty System". As a means of protection against floods, it recommended levees as essential to prevent destruction to life and property by overflow. The Commission was not agreed as to the value of levees as an auxiliary to a plan of channel improvement.

The first appropriation for the improvement of the Mississippi River in accordance with the plans of the Commission was made in March, 1881, and work continued for several years, but several facts were soon developed, namely, that the light contraction works originally designed were much too weak; that the cost of the improvement would be much greater than had been originally estimated; that one of the first results of contraction work was to increase bank caving; and that generally bank revetment was of greater relative importance than had at first been considered.

The work was interrupted in 1885 by a legislative proviso that no works of bank protection or revetment should be executed until it was found that permeable contraction works would not secure the desired stability of the river banks. This prohibitive clause was not revoked until late in 1888 so that four successive working seasons were practically lost. After that date, bank protection became of prime importance, contraction works being restricted to the repair and completion of work under way.

Although the Commission in its report of 1880 had recommended levees as a means of protection against floods, Congress was then unwilling to expend Government funds in protecting the land of private owners, for the first Appropriation Act contained the following proviso:

"That no portion of the sum hereby appropriated shall be used in the repair or construction of levees for the purpose of preventing injury to land by overflow or for any other purpose whatever except as a means of deepening or improving the channel of said river."

In taking this position Congress was apparently influenced by the fact that in 1850 it had, through the passage of the Swamp Land Act, conveyed to the States a free title to all the unsold swamp and overflow lands in the delta basin with the understanding that they would be sold and the proceeds used to protect them against overflow. In the Appropriation Act of 1882 it was again provided:

"That no part of this appropriation shall be expended for repair or to build levees for the purpose of reclaiming land or preventing injury to land by overflows; provided that the Commission is authorized to repair and build levees if in its judgment it should be done as a part of its plan to afford ease and safety to the navigation and commerce on the river and to deepen the channel."

Under the strict prohibitive clause in the Act of 1881 no part of the appropriation was allotted to levees. Under the Act of 1882 which gave somewhat wider discretion to the Commission, a large part of the appropriation was allotted to levee construction. From that time on the policy of allotting money for levee purposes was followed, but it was not until 1890 that the prohibitive clause was removed.

From 1890 to 1917, the periodic appropriations made by law for expenditure by the Commission were divided between levees, revetments, and dredging, but the work on the levees was looked upon as an adjunct to river improvement. It was not until the Flood Control Act of 1917, that flood control became by law as definitely a part of the Commission's work as is river improvement. That Act provided for co-operation by local interests, they to provide the necessary right of way and to pay not less than one-third of the cost of levee construction, and to assume the entire cost of maintenance after completion. The cost of river improvement (revetment and dredging) was to be paid for entirely out of Federal funds.

Considered as an adjunct to river improvement, it was natural that levees should be placed fairly close to the bank. This placed them on the highest ground, thereby reducing their cost, it prevented cross-currents and consequent fill within the river banks during flood, to the detriment of the low-water channel, and helped to stabilize the main channel of the river. Even from a flood-control standpoint there is much to be said in favor of placing the levee line fairly close to the river bank.

Levees

The levee section of 1882 was a modest earth embankment with an 8-ft. crown and slopes of 1 on 3, with practically no free-board above the flood of that year. At Lake Providence this resulted in a section that contained 161 sq. ft., or 31 000 cu. yd. per mile. As the levee lines were extended and the flood flows more and more confined, it became necessary to raise the levee grade and correspondingly increase the section so that, in 1896, there was at Lake Providence a cross-sectional area of 530 sq. ft. and a yardage of 103 000 per mile, and, in 1914, a cross-sectional area of 2 158 sq. ft. and a yardage of 422 000 per mile.

This gradual building upward, toward the levee grade required to confine a given flood completely, is believed to be responsible for the widespread fallacy that when a sedimentary stream like the Mississippi is confined between levees, levee grades must be steadily raised to keep pace with a rise in the river bed. In some instances it has been asserted that the bed of the Mississippi River, due to the levees, has risen until it is above the surrounding country. This view, that there has been a marked change in the elevation of the river bed, is not borne out by the results of surveys made at many different points over a long term of years.

In 1924, a study was made of a reach of the river along the Lower St. Francis Basin in Missouri and Arkansas. This basin has a river frontage of 236 miles, from New Madrid, Mo., to Helena, Ark. The construction of modern levee systems in that section was not begun until 1893, except for about 60 miles on the east bank of the river. The period considered was from 1882 to 1921, inclusive, a period extending from no levees to complete levees. The low-water troughs of each year passing the gauges in the reach were tabulated to show the elevations of such troughs at the various gauges, all of which had been established prior to 1882 and had been read regularly since that date. This comparison showed a decided lowering of the low-water plane.

Recently, more than 2 000 measured cross-sections at different points on the river were compared to show the changes that had occurred between early surveys and those of recent years, and the results show uniformly a moderate but well-defined increase in width, depth, and cross-sectional area. It is a well-recognized fact that Mississippi River bars do build up on a rising river and scour out on a falling river, but these are seasonal changes and the effects are not permanent. Bars also travel down stream, diminishing the depth on one section and increasing it on another; but there is no evidence whatever in the records of the Mississippi River Commission to support the view that the bed of the river is being raised due to the construction of levees.

Prior to the flood of 1927 the major flood of record was that of 1912, and, as a result of that flood, the Commission established levee grade lines along the main river computed to be high enough to contain with a 3-ft. free-board a flood of equal magnitude. At the same time it established new standard levee sections defined as follows:

"Levee: Crown to be 8 ft. wide—reduced to 6 ft. above mouth of Missouri River; front slope to be 1 on 3 to intersection with natural ground (or bottom of old borrow-pits where enlargements have thrown the toe of the levee so far forward); and back slope to be 1 on 3 to intersection with top of banquette, which will be from 5 to 8 ft. below the levee crown.

"Banquette: Top to be 20 ft. wide for levees from 10 to 13 ft. high, 30 ft. wide for levees from 13 to 16 ft. high, and 40 ft. wide for levees exceeding 16 ft. in height; top to have slope of 1 on 10 away from the levee; and rear slope to be 1 on 4 to intersection with natural ground. In cases of extreme height —crossing sloughs or bayous—a false berm may be required beyond the rear slope of the banquette."

Subsequently, slight modifications were authorized by the Commission in both grade and section but, generally speaking, both were substantially unchanged at the time of the flood of 1927. Although work had been in progress

for thirteen years, two more years would have been required for completion of the levee project at the rate at which funds were being appropriated.

The Annual Report of the Commission for 1927 shows a total expenditure to date for levees by the United States from Federal funds of $71 000 000 and an expenditure by the United States from funds contributed by local levee districts of $15 000 000. In addition to this expenditure of $86 000 000 by the United States, State and local organizations, prior to and since the creation of the Mississippi River Commission, expended approximately $152 000 000 on levees and in expenses incident thereto, such as rights of way, repair work, high-water expenses, etc. This combined expenditure of $238 000 000 is represented by 1 880 miles of levees containing approximately 500 000 000 cu. yd. of material. This shows an expenditure of slightly less than 50 cents per cu. yd. of material now in the levees; the difference between this and actual cost per cubic yard being represented by lost or abandoned levees, by expenses for maintenance and repair, and by the cost of closing crevasses and fighting floods.

When Commission reports are consulted, progress toward completion of the 1914 levee project seems slow and the amount of levee lost or abandoned seems large. One reason for the rate of progress lies in the fact that the Commission's jurisdiction on tributaries has been gradually extended, so that, while in 1918 the estimated final contents of the levee line was 473 000 000 cu. yd., in 1927 it was 534 000 000 cu. yd., although no change in grade or section had been made.

In six years, from 1918 to 1924, the lost or abandoned yardage amounted to 15 800 000, about 0.7% per annum of the average annual contents of the system. The losses have been due partly to the necessity of abandoning old lines where threatened by caving banks and partly to changes in alignment in order to secure better foundations. The caving of banks is much the greater cause of losses, and bank revetment is the only remedy for this evil.

Bank Revetment

A number of years ago it was estimated that, of the 2 000 miles of bank-line between Cairo and the Head of the Passes, more than 700 miles were in need of revetment. The funds available for such work, however, have never been sufficient to justify undertaking the work on a large scale or in a systematic manner. Each year it has been necessary to confine revetment work to cases of the most urgent necessity, such as threatened cut-offs or endangered levees that could be replaced only at excessive cost. Even if the work has of necessity been prosecuted in this haphazard manner there was in place, at the end of the fiscal year 1927, 128 miles of revetted bank.

The revetment in most common use is made of small willow trees from 2 to 6 in. in diameter and from 30 to 50 ft. long, which grow rapidly and in abundance on the sand-bars along the river. In the earlier forms, the revetment was composed of these small trees, laid diagonally, in two layers, and the mass was given a certain rigidity and strength by means of large "poles", below and above, tied together by galvanized wire. This type has given way to the

"fascine" mattress, composed of small bundles of young willows bound by galvanized wire. These fascines are placed side by side to make up the length of the mattress, and are held in position by being tied underneath by means of galvanized "sewing strands" running over and under the bundles to longitudinal cables. In the earlier forms of mattress, these fascines were made complete, and were bound in the circular form by short lengths of wire, before being placed in the mattress. In the present mattress, the two processes are combined. Longitudinal wire cables and larger trees or "poles" give the requisite strength and rigidity.

These mattresses are built on floating ways in lengths of 1 000 ft. and of varying widths depending upon the character of the bank to be revetted. The mattresses are sunk in position by stone dumped from barges. In certain places where rock was costly, concrete ballast blocks weighing about 65 lb. were used instead of rock.

A number of different types of reinforced concrete mattresses have also been used, one of them, developed in the Third Mississippi River Commission District, having been in use about five years. This type of revetment is fabricated in units 4 ft. wide by 25 ft. long and 3¾ in. thick, each unit consisting of 25 concrete slabs 1 ft. by 4 ft. by 3¾ in., reinforced and held together by wire reinforcement. These units are cast on deck barges and, as soon as the concrete has hardened sufficiently, a second layer is cast on top of the first, the layers being separated by heavy paper. These units are joined by cables into mattresses on floating launching ways and are sunk with their long side perpendicular to the bank. The willow mattresses are constructed with the long edge parallel to the bank. Since its inception this type of revetment has been improved by increasing the size and, therefore, the life of the reinforcement and by increasing the thickness of the slabs from 3 to 3¾ in., but sufficient time has not elapsed to determine fully the relative durability of concrete and willow mattresses. The concrete mattresses, being smaller in size and with the long axis perpendicular to the bank, more readily adjust themselves to inequalities in the river bank than the longitudinal willow mattresses and, furthermore, they are nearly 20% cheaper.

For certain reaches of the river this type of concrete revetment is believed to be too light. For use in such places experiments are in progress (1927) with a mattress composed of overlapping and interconnecting concrete slabs 4 in. thick, 6 ft. wide, and 11 ft. long.

Mattresses, whether of willow or concrete, protect only the under-water slope, extending out from the water line to about 50 ft. beyond the foot of the slope. Above the water line the bank is graded to a slope of 1 on 3 and paved with rip-rap stone, or concrete.

To the end of the last fiscal year (1926) the Commission had expended in new revetments, and in repairs and renewals, $61 000 000, an average of $475 000 per mile of effective revetment. In addition to its expenditures in levees and revetments, the Commission is maintaining and operating a fleet of dredges at an average annual expenditure of $600 000, in order to maintain a navigable depth of 9 ft. over bars during low-water periods.

As revetment work progresses the volume of material annually caved into the river will decrease, and it is anticipated that this will greatly improve the low-water channel and lessen expenditures for dredging.

The levee work is primarily in the interest of flood protection and, as was mentioned previously, is paid for in part by local interests. The dredging work is in the interest of navigation and, like most other navigation projects, is paid for wholly by the Federal Government.

The revetment work on the main river is essential to both flood protection and to navigation, but under existing law it is regarded as part of the navigation project and paid for wholly by the United States, except in one or two places where local levee districts have voluntarily paid for such work in order to supplement what the Commission was able to do.

In appraising the work of the Mississippi River Commission it should be borne in mind that it has been working toward a limited objective at a rate fixed by Congress. It was never assumed by the Commission, nor by Congress, that its plan would do more than protect against a flood equal to that of 1912. In working toward this limited objective it has many times been importuned to provide special relief for New Orleans by the construction of one or more spillways above or below the city so that it might be spared the expense of raising its wharves to a height compatible with the 1914 grade line established for the entire river. Such work the Commission has heretofore declined to undertake on the ground that the limited funds at its disposal should be used in giving a uniform measure of relief throughout the valley and that to divert $5 000 000 or $10 000 000 to the construction of spillways at or near New Orleans would be unfair to those levee districts the levees of which were still not up to the grade line of 1914.

Flood of 1927

The rainfall throughout the valley of the Mississippi River which caused the flood of 1927 was extraordinary in volume and in duration and it occurred at a time when the natural storage in the Mississippi itself and the storage basins contiguous to it in the delta were already well filled. The rainfall in Eastern Oklahoma, Western Arkansas, and Southern Missouri was unprecedently heavy and caused discharge from the Arkansas River almost as great as that of ordinary high water in the Mississippi itself.

It is difficult to compare the magnitude of different Mississippi river floods at a given point on the basis of maximum measured discharges alone, for at the same gauge height the discharge will vary widely due to differences in slope, and the slope, in turn, will depend upon the extent to which the river basin is filled and upon the rate of rise.

Columbus, Ky., 22 miles below the mouth of the Ohio River, at Cairo, is a gauging station, and discharge measurements made there or in that vicinity are available for each flood-year since 1892. As a check on the observed discharges it is customary to assemble the volume contributed by the Mississippi and the volumes contributed by the Ohio at Evansville, Ind., and its principal tributaries below that point. Table 13 shows the volume of discharge

(as scaled from rating curves) contributed by tributaries of the Ohio and by the Ohio River as a whole. The greatest discharge that occurred within a period of 3 to 6 days before the flood crested at Cairo was used for all stations. The discharge at the maximum recorded stage for each station is also given.

TABLE 13.—VOLUME OF WATER CONTRIBUTED BY OHIO RIVER AND ITS TRIBUTARIES, IN CUBIC FEET PER SECOND.

River.....	Wabash.	Ohio.	Cumberland.	Tennessee.	Ohio.*
Station.....	Mt. Carmel, Ind.	Evansville, Ind.	Nashville, Tenn.	Florence, Tenn.	Total.
Year.					
1890	140 000	572 000	176 000	341 000	1 229 000
1892	90 000	471 000	114 000	207 000	882 000
1893	190 000	475 000	34 000	185 000	884 000
1897	143 000	356 000	166 000	575 000	1 240 000
1898	286 000	615 000	52 000	126 000	1 079 000
1903	184 000	551 000	113 000	218 000	1 016 000
1904	283 000	501 000	79 000	131 000	994 000
1907	171 000	623 000	72 000	52 000	918 000
1912	140 000	552 000	148 000	270 000	1 110 000
1913	346 000	665 000	146 000	238 000	1 395 000
1916	164 000	526 000	72 000	107 000	869 000
1917	52 000	550 000	102 000	290 000	994 000
1920	76 000	551 000	64 000	178 000	869 000
1922	190 000	560 000	109 000	278 000	1 137 000
1927	100 000	435 000	79 000	200 000	814 000
Maximum recorded..	400 000	673 000	235 000	596 000	.....

* This column obtained by summation of quantities shown in the preceding columns, and not by measurement.

In Table 14, the Ohio River contributions are added to those of the Mississippi at St. Louis and the total compared with the maximum observed discharge at Columbus, as well as with the discharge at Columbus as scaled from rating curves. For the Mississippi River contribution there was used the highest gauge reading which occurred at St. Louis within a period of 2 to 3 days before the flood crested at Columbus.

The sum of the discharges at St. Louis and from the Ohio River does not represent the total discharge at Columbus, because it does not include the run-off from the drainage basins of Meramec, Kaskaskia, and Big Muddy Rivers and a number of smaller streams.

Table 14 shows that the maximum observed discharge at Columbus in 1912 and 1913 was nearly 300 000 sec-ft. more than was measured in 1927, yet the maximum gauge reading in 1927 was 1.6 ft. higher than in 1913. These differences are believed to be due in part to differences in the shape of the flood crests and more particularly to an incomplete levee line in 1912 that induced a slope and velocity that were abnormal at the time the discharge was measured. The discharge at Columbus obtained by summation of the flow of contributing streams was 1 702 000 sec-ft. in 1912 and 1 614 000 sec-ft. in 1927. On the other hand, if dependence be placed in the rating curve, the 1927 flood exceeded the 1912 flood at Cairo by 67 000 sec-ft. In formulating plans for protection

against floods of the future, it will be conservative and on the side of safety to assume a discharge corresponding to a foot on the gauge at or near maximum stages, no greater than that shown by past floods. In the 1927 flood a foot on the gauge at and near the maximum stage corresponds on the average to a flow of 60 000 sec-ft., and, as no flood of the past shows a smaller average discharge, that ratio has been used by the Commission in its present plans and estimates.

TABLE 14.—DISCHARGE, IN CUBIC FEET PER SECOND.

Year.	Mississippi River, at St. Louis, Mo.	Ohio River, from Table 13.	Sum of Columns (2) and (3).	Mississippi River, at Columbus, Ky.	
				Scaled from rating curves.	Maximum observed.
(1)	(2)	(3)	(4)	(5)	(6)
1890	72 000	1 229 000	1 301 000	1 312 000	
1892	408 000	882 000	1 290 000	1 332 000	1 401 000
1893	601 000	884 000	1 485 000	1 470 000	1 528 000*
1897	283 000	1 240 000	1 523 000	1 464 000	1 462 000
1898	283 000	1 079 000	1 362 000	1 393 000	1 517 000
1903	377 000	1 016 000	1 393 000	1 426 000	1 483 000
1904	383 000	994 000	1 377 000	1 389 000	1 502 000†
1907	429 000	918 000	1 347 000	1 433 000	1 543 000
1912	592 000	1 110 000	1 702 000	1 733 000	2 015 000‡
1913	323 000	1 395 000	1 718 000	1 750 000	2 015 000§
1916	617 000	869 000	1 486 000	1 497 000	1 775 000
1917	285 000	994 000	1 279 000	1 362 000	1 420 000
1920	446 000	869 000	1 315 000	1 422 000	1 527 000
1922	311 000	1 137 000	1 448 000	1 520 000	1 501 000
1927	800 000	814 000	1 614 000	1 300 000	1 728 000‖

* Another measurement at same stage gave 1 434 000 sec-ft.

† Another measurement at about same stage gave 1 447 000 sec-ft.

‡ Measured at 48.5-ft. stage. At the maximum stage the discharge measured 1 854 000 sec-ft.

§ Mean of seven consecutive measurements was 1 959 000 sec-ft.

‖ Cairo gauge was 53.7 ft. at time of observation.

The maximum discharge past the latitude of Arkansas City in April, 1927, has been estimated at 2 662 000 sec-ft. This is 250 000 sec-ft. more than in 1912 and about 200 000 sec-ft. more than in 1916.

The estimated maximum outflow from the latitude of Old River was 2 350 000 sec-ft., about 300 000 sec-ft. less than at Arkansas City. If to this difference the flow of the Yazoo and the Red Rivers is added, a measure of the regulating influence of storage in the lower ends of the Yazoo and Tensas Basins, and in the river itself, is obtained.

Subsequent to the flood of 1927 the Commission was directed by the Secretary of War and Chief of Engineers, U. S. Army, to revise its plans to conform to the new conditions, and the Commission has submitted a report on that work to the Chief of Engineers. The writer will outline herein in a general way the nature of the problem and refer briefly to some of the methods considered.

As a basis for a new project, it was determined to set up a probable future maximum flow at Cairo. The discharge at Cairo in 1927 was approximately 1 800 000 sec-ft. In determining how much larger a flood should be provided for, consideration was given to the fact that if to the maximum discharge of the Mississippi at St. Louis there was added the maximum discharges of the

Wabash at Mt. Carmel, the Ohio at Evansville, the Cumberland at Nashville, and the Tennessee at Florence, the total would aggregate more than 3 000 000 sec-ft. If the maximum discharge of the Ohio and the maximum discharge of the Mississippi at Cairo were added, the total would be about 2 700 000 sec-ft. If to the estimated discharge of the 1927 flood (1 800 000 sec-ft.), there be added the difference in flow, at flood stages, of the Ohio in 1913 (maximum) and in 1927, the total will be 2 400 000 sec-ft.

TABLE 15.

Miles below Cairo, Ill.	Station.	New levee grade.	Existing levee grade.	Increase in levee heights, in feet.
0	Cairo, Ill.	70.4	58.0	12.4
22	Columbus, Ky.	64.7	52.7	12.0
71	New Madrid, Mo.	56.5	47.6	8.9
124	Cottonwood Point, Mo.	53.6	46.0	7.6
175	Fulton, Mo.	54.4	47.0	7.4
227	Memphis, Tenn.	58.4	51.5	6.9
273	Mhoon Landing, Miss.	60.1	50.0	10.1
307	Helena, Ark.	71.0	58.5	12.5
354	Sunflower Landing, Miss.	72.6	56.0	16.6
392	Mouth of White River.	78.4	61.9	16.5
437	Arkansas City, Ark.	79.5	60.5	19.0
480	Greenville, Miss.	72.8	55.8	17.0
543	Lake Providence, La.	69.1	53.7	15.4
602	Vicksburg, Miss.	72.0	58.0	14.0
662	St. Joseph, Mo.	68.7	54.0	14.7
706	Natchez, Miss.	72.7	57.5	15.2
773	Red River Landing, La.	69.5	57.5	12.0
807	Bayou Sara, La.	61.8	51.5	10.3
841	Baton Rouge, La.	57.9	48.1	9.8
862	Plaquemine, La.	52.1	43.7	8.4
894	Donaldsonville, La.	46.4	39.0	7.4
911	College Point, La.	40.6	34.4	6.2
965	Carrollton, La.	30.8	25.2	5.6
1 047	Fort Jackson, La.	14.4	11.0	3.4

All these combinations may be classed as "possibilities", but it does not seem probable that rainfall sufficient to produce such coincident floods will ever occur. In its determination of this question the Commission gave full consideration to the views of the local Weather Bureau meteorologist at Cairo and to the views of the Chief of the U. S. Weather Bureau. For the purposes of this paper it is proposed to assume 2 250 000 sec-ft. as the probable maximum flood of the future at Cairo. If the relation between gauge and discharge established in the 1927 flood were maintained, such a flood, confined by levees, would reach a stage of 65 ft. on the Cairo gauge. This assumed discharge is 25% greater than that of 1927, a factor of increase relatively much larger than that assumed in the Miami Conservancy project, when the relative magnitude of the two drainage areas is considered. If protection against such a flood were provided by levees alone, it would require an increase in levee heights as shown in Table 15.

This increase in levee height is based upon a 5-ft. free-board instead of the 3-ft. free-board heretofore used, a modification that was generally recommended by the engineers of local levee districts as a result of their experience with the 1927 flood. These levee engineers were also practically unanimous in recommending that the levee section be strengthened by flattening the

river-side slope from 1 on 3 to 1 on 4, by increasing the crown width from 8 to 12 ft., and by flattening the land-side slope so as to contain completely a saturation line of 1 on 7 springing from a point on the river-side slope 2 ft. vertically below the crown. The flattening of the river-side slope was advocated largely as a means of protection against wave wash. The increased free-board and increased width of crown were advocated as increased factors of safety and the flattening of the land-side slope was for the purpose of reducing seepage during long-continued high stages. To increase the levees to such heights and to increase the section as outlined would require an expenditure for main river levees alone of more than $500 000 000.

The Commission in revising its project, considered all the many methods that have been suggested as substitutes for or as adjuncts to a levee system. Many of them have been covered by other papers and to include herein a discussion of them would result in duplication. However, even at the risk of duplication it seems desirable to refer briefly to reservoirs and to spillways and outlets.

Reservoirs

High floods in the Mississippi, as far as records are available, have crested at Cairo between February 22 and May 9, except the flood of 1916. The flood wave at Vicksburg is much longer and flatter than at Cairo and at this point the river is within 4 ft. of its crest for a period varying from 20 to 80 days. The length of the Mississippi River flood crest, the great distance of the head-water reservoirs from the region of the flood, and the difficulty of forecasting these floods, indicate that head-water reservoirs above Cairo to be effective must store water for a period of about 90 days. Data concerning 245 proposed reservoirs were examined and from the storage graphs there was determined the percentage of reservoir capacity which would have been utilized each flood-year.

A first study concerned head-water reservoirs above Cairo and assumed them to be operated solely in the interest of Mississippi River flood reduction. Reservoirs having an aggregate capacity of 71 000 000 acre-ft. were considered, the cost per acre-foot ranging from 50 cents to $112. A storage-cost curve was prepared based on storage at a uniform rate during the storage period, on an efficiency as measured by the quantity of water stored equal to 75% of reservoir capacity, and on a relation between gauge and discharge at Cairo of 1 ft. equals 60 000 sec-ft. From this cost curve it appeared that the cost of reducing flood heights at Cairo would range from $62 000 000 for the first foot to $230 000 000 for the fourth foot, and that a reduction of 4 ft. by means of such reservoirs, equivalent to 240 000 sec-ft., would cost $560 000 000.

A more detailed analysis was then made using actual rates of storage instead of assumed uniform rates and actual efficiency of reservoirs instead of an assumed average of 75 per cent. Allowance was also made for the effect of storage in the river valleys between the reservoirs and Cairo. This more detailed study was limited to the Ohio Basin and comparison made between the floods of 1913 and 1922. The effect of valley storage was determined approximately by showing graphically the relation between the inflow

into the Ohio River from its tributaries and the outflow as measured near its mouth (Figs. 2 and 3). The inflow from the discharge diagrams of tributaries having a total drainage area of 170 100 sq. miles, has been increased by direct ratio to allow for an additional drainage area of 33 800 sq. miles. The composite inflow graph of all the tributaries was synchronized with respect to the outflow graph by taking into consideration the time of travel of water from the mouth of each tributary to the mouth of the Ohio. The difference between the inflow and the outflow curve appears to be a direct measure of the effect of valley storage and the true effect at Cairo of reservoir storage in the Ohio Basin can only be determined by taking into consideration the modifying influence of valley storage. The 1913 flood in the Ohio Valley was characterized by a sharp peak, whereas that of 1922 was a long low flood. In the latter, valley storage was a much less important factor than in the former.

The cost and effect of reservoirs was analyzed also on the assumption of joint use, that is, a combination of Mississippi River flood control with local flood control, the development of power, and improvement for navigation, or by a combination of three or more uses. A great many such combinations are possible and although the several uses are more or less conflicting, it may be said generally that such joint use will result in some reduction of cost per acre-foot chargeable to Mississippi River flood control.

A group of the most desirable reservoirs above Cairo, as determined by the ratio of benefits to costs, was then selected for further study. This group had a capacity of about 38 000 000 acre-ft., the average gross cost being between $10 and $11 per acre-ft., and the net cost chargeable to Mississippi River flood control between $7 and $8 per acre-ft. To reduce flood heights at Cairo by this selected group would require a gross expenditure for the first foot of reduction of $104 000 000, or a net expenditure chargeable to the Mississippi River of $40 000 000, assuming that the other interests benefited could be induced to pay the difference. The second foot of reduction would cost $160 000 000, of which it is estimated that $17 000 000 could equitably be charged to other interests. The average effect of this group in reducing gauge heights at Cairo during past floods would have been 2.65 ft., and had the entire group been built the average net cost under the assumptions made would be $115 000 000 per ft. of reduction on the Cairo gauge, or nearly $10 000 000 per in. of reduction. Expressing the reduction in terms of discharge, the cost would average $190 000 000 per 100 000 sec-ft. for a maximum reduction of 159 000 sec-ft.

Suggested reservoir sites on the St. Francis, White, and Arkansas Basins were also considered. These reservoirs are generally located in areas of heavy rainfall, which are important contributors to floods in the Lower Mississippi, and the proposed reservoirs are of such size that their capacity could be utilized effectively during most Mississippi River floods. These reservoirs have a gross capacity of 25 000 000 acre-ft. and are estimated to cost $215 000 000, or $8.50 per acre-ft. This cost may be reduced under a joint-use plan; furthermore, by reason of their proximity to the delta they should be more valuable than an equal amount of storage 1 000 miles above Cairo.

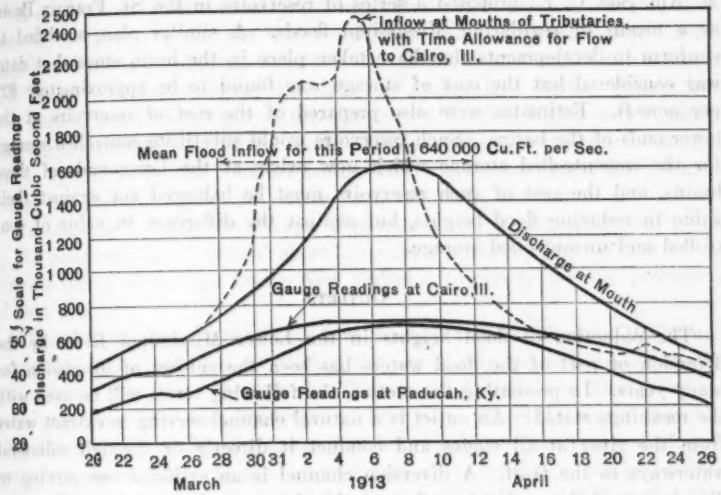

FIG. 2.—INFLOW AND OUTFLOW OF OHIO RIVER DURING 1913 FLOOD.

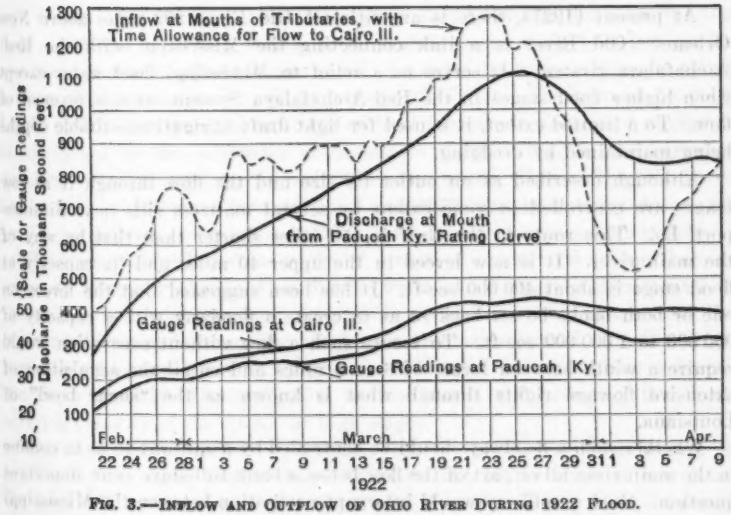

FIG. 3.—INFLOW AND OUTFLOW OF OHIO RIVER DURING 1922 FLOOD.

Studies were also made of suggested reservoirs in the St. Francis, Yazoo, and Tensas Basins. About thirty years ago the late James A. Seddon, M. Am. Soc. C. E., proposed a series of reservoirs in the St. Francis Basin as a means of regulating Mississippi floods. A similar plan, modified to conform to developments that have taken place in the basin since that date, was considered but the cost of storage was found to be approximately $27 per acre-ft. Estimates were also prepared of the cost of reservoirs at the lower ends of the basins. Such reservoirs would substitute controlled storage for the uncontrolled storage which now exists at the lower ends of these basins, and the cost of such reservoirs must be balanced not against their value in reducing flood heights, but against the difference in value of controlled and uncontrolled storage.

Outlets

The reduction of flood heights in the Lower Mississippi River by the diversion of part of the flood waters has been the subject of discussion for many years. In presenting the matter, the following terms will be used with the meanings stated: An outlet is a natural channel serving to extract water from the river at all stages and conduct it directly or through collateral waterways to the Gulf. A diversion channel is an artificial one serving to divert part of the main river flow to the river at a lower point or by independent collateral channels to the Gulf, the flow into it being wholly or partly under control. A spillway is a structure designed to afford regulated entrance of flood waters into a diversion channel.

At present (1927), there is an outlet at Old River 202 miles above New Orleans. Old River is a link connecting the Mississippi with the Red-Atchafalaya System. It serves as a relief to Mississippi flood water except when higher flood stages in the Red-Atchafalaya System cause a reversal of flow. To a limited extent, it is used for light-draft navigation, suitable depths being maintained by dredging.

Although described as an outlet its size and the flow through it at low stages are controlled to some extent by several mattress sills near Simmesport, La. This route to the Gulf is 150 miles shorter than that by way of the main river. It is now leveed in the upper 40 miles and its capacity at flood stage is about 400 000 sec-ft. It has been suggested that the levees on one or both banks be set back so as to create a floodway with a capacity of 900 000 to 1 000 000 sec-ft. To secure such a flow without excavation would require a width between levees of 6 to 8 miles and entail the acquisition of extensive flowage rights through what is known as the "sugar bowl" of Louisiana.

Whether such a floodway should be controlled by a spillway so as to confine in the main river all or part of the flow below a bank-full stage is an important question. Such a spillway would interrupt navigation between the Mississippi and the diversion channel unless a lock were also provided. On the other hand unless controlled, so large a diversion may cause silting in the main

channel to such an extent as to interfere with navigation at low stages and to diminish its flood-carrying capacity at high stages.

A diversion channel through the Tensas Basin with capacity ranging from 600 000 to 750 000 sec-ft. was also considered. Such a channel would leave the main river at or near the mouth of the Arkansas and extend to the upper end of the proposed floodway through the Atchafalaya Basin. Study was also made of a diversion channel through the St. Francis Basin, with capacity ranging from 300 000 to 500 000 sec-ft. Unless such a channel were extended to the White River and thus connected with the Tensas Basin diversion, it would not permanently divert water from the main channel.

In addition to these three major diversion plans, consideration was given to the construction of several short diversions with capacity of 250 000 to 300 000 sec-ft. above and below New Orleans, primarily for the protection of that city.

Diversion of part of the flood flow through the St. Francis Basin will apparently cost more than to confine the entire flow between levees on the main river. Diversions through the Tensas and Atchafalaya Basins are far more promising from the standpoint of first cost. Even if cheaper than the plan which would confine the entire flow between levees, the creation of such secondary channels for the discharge of part of the flood flow can be justified only if the opponents of diversion channels are mistaken in their belief that they will not permanently lower flood heights.

Opponents of outlets use as an illustration the changes in the relation of stage to discharge at Red River Landing, a short distance below the Atchafalaya Outlet. The records of the Mississippi River Commission show for the flood of 1882 a measured discharge of slightly more than 1 500 000 sec-ft. at a stage of 48.5 ft. In 1912, practically the same discharge was measured at a stage of 53.2 ft. and, in 1927, at a stage of 57.5 ft. The increase in flood height of 9 ft. between 1882 and 1927 without increase in volume of discharge is attributed by the opponents of outlets to deterioration in the carrying capacity of the channel of the main river due to the effect of the Old River Outlet. (The difference between 1882 and 1912 may be entirely accounted for by the difference in slope in the two cases caused by crevasse effect below.)

Evidence obtained from a study of crevasses supports the view that an outlet or secondary channel that diverts water at all stages is detrimental to the main stream, and that the larger it is in proportion to the main stream, the greater will be the deteriorating effect upon the main channel. Such a diversion tends to cause a raising of the bed below the outlet, thereby reducing the depth available for navigation at low stages and reducing the flood-carrying capacity of the stream at high stages.

Opponents of controlled diversions, even if the diversion be limited to over-bank flow, argue that if water be taken out of the river at any point the level will be lowered at that point and the slope of the river increased above the point of diversion and reduced below that point; that the increased velocity above will increase the silt-carrying capacity of the river, and that

it will approach the spillway with a greater load of silt than it would have had under natural conditions; that the reduced slope and reduced velocity below the spillway will result in sedimentation and reduction in the cross-section of the river; that this will continue until the river is again able to carry off its portion of the sedimentary matter; that the necessary increase in velocity can only be obtained by increased head; and that increased head means an increased flood level.

This belief, that the Mississippi River is so charged with sedement that the reduction in slope and velocity below a controlled diversion channel will cause deposits, is not borne out by the findings of Humpreys and Abbot. Furthermore, if the diversion is so built that it will not come into action until after the river reaches a bank-full stage (approximately 1 000 000 sec-ft.) and so controlled that the flow through it can be reduced as soon as the crest passes, the changes in slope and velocity can be kept within such narrow limits that deposits should not occur. Even if some sedimentation does take place, it should be completely removed later by the river itself, if the spillway is closed as soon as the discharge of the river falls within the capacity of the main channel.

Conclusion

Protection against an assumed probable maximum flood at Cairo of 2 250 000 sec-ft. can be provided by enlarging and strengthening the existing levees, but, no matter how well the levees are built, nor how well they are protected by revetment of the banks, an 1 800-mile levee line of the height necessary to contain such a flood would carry with it a menace to the people living behind it that should be avoided if practicable.

Head-water reservoirs above Cairo as a means of reducing levee heights will apparently cost very much more than a series of diversion channels or floodways through the delta basins.

Some of the proposed diversions will apparently justify themselves from an economic standpoint in that they will cost less than corresponding increase in levee heights; others will cost more, but may be justified by reason of the greater safety that they will afford the inhabitants.

Such secondary channels are not substitutes for levees on the main river. Even with all the proposed diversion channels built, the existing levees will have to be maintained and even increased in height in order to provide protection against the assumed flood of 2 250 000 sec-ft., at Cairo.

Whatever may be assumed as the probable maximum flood of the future, an even greater flood may some day occur, sufficient to overtop the levee line. To provide for such a contingency, it has been proposed to construct along the crown of the levee a series of over-bank spillways that would be brought into play only when overtopping was imminent. Such structures would provide the means of substituting a minor disaster for a major one and permit prompt recovery from the effects of such a super-flood.

In the writer's opinion, the most dependable, as well as the cheapest, method of protecting the delta basin of the Mississippi River against a

probable future flood lies in a system of main river levees, supplemented by a series of controlled diversion channels through which excess flood waters can be carried to the Gulf.

As an integral part of such a flood-control plan, the main river should be stabilized by revetting the caving banks. While such stabilization is regarded as an essential part of the flood-control project, it will also greatly benefit navigation at low stages.

FOREST COVER AS A FACTOR IN FLOOD CONTROL

By E. F. McCarthy,* Esq.

In an address before the Mississippi Flood Control Conference at Chicago, Ill., early in June, 1927, United States Forester W. B. Greeley took the position that, while the main reliance for handling large flood discharges must be placed upon engineering structures, forests have a definite part in flood control, together with other forms of land use which check erosion and favorably influence natural storage conditions. This attitude is the rational one, and engineers facing the problem of flood control in the Mississippi will doubtless feel that any agency which will assist in lowering the flood crests and in retaining the flood water within predetermined bounds should be used if economically feasible.

The relation of forest to stream flow is a subject which has received extensive discussion. Several good bibliographies have been collected. One is incorporated in the Final Report of the National Waterways Commission,† and a later one is contained in a report by the California State Board of Forestry.‡ No attempt has been made to repeat references in either of these publications. The Forestry Committee on the Relation of Forests and Waters submitted the following statement to the Fifth National Conservation Congress:§

"In the mountains, the forests break the violence of rain, retard the melting of snow, increase the absorptive capacity of the soil cover, prevent erosion, and check surface run-off in general, thus increasing the underground seepage and so tend to maintain a steady flow of water in streams."

Zon† has summarized the effect of forests on stream flow, as follows:

"Among the factors, such as climate and character of the soil, which affect the storage capacity of a water-shed, and therefore the regularity of stream flow, the forest plays an important part, especially on impermeable soils. The mean low stages as well as the moderately high stages in the rivers depend upon the extent of forest cover on the water-sheds. The forest tends to equalize the flow throughout the year by making the low stages higher and the high stages lower.

"Floods which are produced by exceptional meteorological conditions cannot be prevented by forests, but without their mitigating influence the floods are more severe and destructive."

A study was conducted by the U. S. Forest Service during the summer of 1927 to determine the relation of the forests to floods in the Mississippi.

* Director, Central States Forest Experiment Station, Forest Service, U. S. Dept. of Agriculture, Columbus, Ohio.

† "Forest and Water in the Light of Scientific Investigation," by Raphael Zon, Final Rept., National Waterways Comm., Appendix V.

‡ "Erosion and Flood Problems in California," by E. N. Munns (Rept. by California State Board of Forestry to the Legislature (1921) on Senate Concurrent Resolution No. 27).

§ "The Relation of Forests and Water," by Raphael Zon and others, Sub-Committee on Forest Investigations.

The collection of information relative to the Ohio drainage came under the supervision of the Central States Forest Experiment Station, so that direct reference will be made to this tributary.

The Ohio Basin, which contains approximately 203 230 sq. miles, once practically all forested, now has an estimated forest cover of 68 220 sq. miles. About 33 120 sq. miles of this land is woodland incorporated in the farms, of which about one-third is pastured. These statements have been qualified carefully because exact information in regard to the actual forest cover and its condition has never been collected for this region. In only a few instances, notably in Illinois, has an effort been made actually to map the existing woodland. The best general statistics are those included in the Census of Agriculture for 1925, and even this report does not adequately classify land lying outside the farms so that an accurate judgment can be made as to the amounts of forested and non-forested wild land. The foregoing statement of woodland acreage is based on the Census report, modified by the writer's judgment of the forested acreage of wild land. Of course, all available information which could be procured from State and individual sources has been used. From this summary the Ohio Valley may be considered as one-third forested.

The present trend, especially in the mountainous and hilly sections, is to abandon farm land rather than to extend its present acreage by further clearing. This tendency is so obvious as to attract even casual observation. The causes are chiefly three: (1) The inability of production on rough land to compete with that on farms more favorably situated as to soil, topography, and market; (2) the severe erosion and destruction of the steeper fields; and (3) industrial development and improvement of road communications which is draining the forested area of its labor element.

Many of the broad-leafed species may be found in the forest of the Ohio Valley. The exceptions to this are a comparatively limited acreage of spruce and fir forest in the higher plateau of West Virginia and the Southern Appalachian Mountains, and the mixture of scattered pine and hemlock found in some places with the broad-leafed species. The more exposed ridges also may support small patches of pure yellow pine. From this statement it is obvious that this entire forested area each year receives a carpet of leaf litter, chiefly from hardwoods, which are left largely defoliated during the winter season.

The forest of the Ohio Valley is now very unsatisfactorily stocked. Even in its virgin condition there were large spaces in the forest crown cover. Cutting, fire, and grazing have made further serious inroads on the virgin stand. The older timber now left is only a remnant of the original stand. These influences, in turn, have increased the general undergrowth of rhododendron, laurel, and other shrubs. Significant, in so far as flood prevention is concerned, is the thinness of the stand which now occupies the forested land of this valley. Only in the few places which have received adequate protection from fire for a considerable number of years is there a dense stand of timber with a well-covered forest floor. In the past the rougher mountain and plateau lands have been frequently burned over. In 1927, 211 424 acres

of land were reported burned in Ohio, Indiana, Kentucky, Tennessee, and West Virginia. This represents about 0.7% of the total wooded acreage of these States. The foresters of the Ohio Valley States recognize the necessity for better protection from fire and improved management of timber lands, both for increasing the protective value to their water-sheds and for the yield of timber.

The relation of the forest to run-off is most closely established through its influence on erosion, since it is through the washing away of the finer soil particles that channels are established for the flow of water. The first place to seek control of a flood is at its source where every small impediment is most effective in delaying the union of raindrops into rivulets and these, in turn, into streams strong enough to sweep clear a direct channel. The best that can be done at the source is to secure the most effective percolation of water from the surface to the water-table.

The fact is well established that the forest, through its littered floor and porous soil, does cause impediment to the flow of water on the ground surface and does increase the water storage, thereby maintaining the flow of springs in dry seasons. A large volume of evidence has been accumulated in support of this contention. Several striking examples are cited.

In 1922, Charles G. Adsit, M. Am. Soc. C. E., Vice-President and Executive Engineer of the Georgia Railway and Power Company, stated:

"At our Morgan Falls Plant on the Chattahoochee River, which is below large cultivated areas, the reservior in 1904 covered 750 acres. After ten years' service, there was no reservoir capacity worth mentioning left, it having all silted shut, leaving only a channel through the center of the reservoir for the river flow. On the other hand, our Mathis Reservoir, above which is only forested area, after ten years' service, shows little or no sign of silt whatsoever."

The following statement is made in a recent letter by Edgar P. Kable, General Manager, The York Water Company, York, Pa.:

"Our observations of the effect of forest planting on our water supply in dry times has been that it has increased the dry weather run-off very materially.

"We have a 700-acre tract planted with 680 000 evergreen trees and although we have no gauge at present, nor did we have prior to the building of the dam, yet this fact is noticeable.

"The quality of the water in the stream has also materially improved as there is very little erosion from the banks, and, as an example, when there was a great downpour of rain * * *, the water in this dam remained clear, whereas some of the neighboring streams became very turbid."

The litter-covered forest floor is generally recognized as the best protection against erosion of soil. Through the action of the litter in preventing the packing of soil by rains, the forest soil remains in a loosened condition throughout the season. The soil is loosened by the action of frost, as well as rodents, and other animal life, and is especially porous in the early spring months at the time when the precipitation or accumulation of snow water is greatest. A mere comparison of the forest soil made porous by these agents and by the penetration of tree roots, with the packed soil of adjacent fields,

will convince any one of the value of this cover in retarding run-off. The action of the litter in delaying run-off is most pronounced in the first few rods of movement of the water while it is accumulating in the natural drainage channels.

The terrific loss of soil through erosion is not generally understood, and an excerpt from *Research Bulletin 63* of the Missouri Agricultural Experiment Station* is inserted to bring out this point. Experiments were conducted for a period of six years on carefully controlled plots with an average gradient of 3.68 per cent. In summarizing the results, the authors state:

"If farm land should erode as rapidly as the land in these experiments the surface 7-in. layer would be removed at the following rates: uncultivated land in 29 years; plowed 4 in. deep in 24 years; plowed 8 in. deep in 28 years; planted to corn annually, in 56 years; wheat, annually, in 150 years; rotation, corn, wheat, and clover in 437 years; bluegrass sod in 3 547 years.

"The chemical analyses showed that the amounts of nitrogen, phosphorus, calcium, and sulphur in the eroded material from corn or wheat land may equal or exceed the amounts taken off in the crops. There were only small amounts of nitrogen, as nitrates, lost in the run-off water."

This Missouri experiment brings out the great difference in loss of soil from bare or cultivated land and that protected by bluegrass sod. While it does not afford evidence of the protective value of the forest, it does indicate the loss which might be expected, even on comparatively level land, if a sod cover should fail and leave the land bare.

The most striking example of complete removal of soil in a forest region is afforded by the area damaged by smelter fumes near Ducktown, Tenn. Here, each few feet of surface has developed its water channels and tremendous quantities of the surface soil have been washed into the tributaries of the Hiwassee River. The significant thing in connection with the Ducktown area is that the soil accumulated during ages, under the protection of a forest cover, was eroded in a few years when exposed by the damage from smelter fumes.

The dry weight of hardwood litter annually deposited on the forest floor has been estimated at about 2 tons per acre. This will vary somewhat according to the species and the density of the stand. The rapidity of disintegration also varies, as influenced by wind and weathering. For use in this paper, determinations were made on plantations with an average age of 17 years, located at the Ohio Agricultural Experiment Station, at Wooster, Ohio. The work was done in the week ending October 8, 1927, before the leaf crop had fallen. The litter on sample areas was weighed and its moisture content determined. The dry weight of litter per acre on these plantations varied from 1 827 lb. to 17 545 lb. and is able to absorb 1½ to 2½ times its dry weight in water. It is notable that the litter underneath the soft woods, pines, spruce, and Douglas fir, exceeds that of the hardwoods at this time of year.

The relative influence of forested and non-forested areas on run-off and erosion has been specifically studied at several places in recent years. The most notable of these, perhaps, is the work being done by the U. S. Forest

* "Erosion and Surface Run-Off Under Different Soil Conditions," by L. F. Duley and M. F. Miller.

Service in co-operation with the U. S. Weather Bureau at the Wagon Wheel Gap Station, in Colorado. Here, two water-sheds as nearly similar as could be found were subjected to measurements for a period of years. One of them was deforested and the measurements were then continued for a further period of years. A preliminary report of this work has been made,* and the final report is now (1927) in preparation. Experience gained in the Wagon Wheel Gap experiment shows the extreme difficulty of securing adequate values through the study of paired water-sheds.

A less extensive study of run-off has been made† by C. E. Ramser, M. Am. Soc. C. E. While attempting to determine the drainage requirements for road purposes, he found the forest an important factor in controlling the rate of run-off. He says of his results, "these values show quite conclusively that timber has a decided influence in reducing the rate of run-off from a water-shed". He further qualified this statement, however, by a discussion of the influence of duration of rainfall on the rate of run-off. His work is further quoted in the Appendix.

A recent paper by Mr. H. Burger,‡ reports a study in Switzerland, where a comparison was made of forested and non-forested water-sheds. He found that the crest of the flood was not only delayed by the forest cover, but that the total run-off per square kilometer was also reduced. In both these instances neither water-shed was completely covered by forest, but a sufficient difference existed to bring out these facts.

The most conclusive study made thus far is that on two water-sheds in the White Mountains.§ In comparing two water-sheds, one largely forested and one deforested and open, the following conclusions have been drawn: At the end of each period of observation there was more snow on the forested than on the deforested basin; the loss of water from snow storage was heavier on the deforested basin; the rate of run-off from the forested basin was less than two-thirds that from the deforested basin; the maximum flood flow from the forested basin never exceeded 71% of that on the deforested basin; and, lastly, considering the precipitation over the entire period, the forested basin added largely to the ground-water storage, whereas the deforested basin drew in large amounts from that storage.

Mr. I. T. Bode,‖ in describing the results of his work in Iowa, claims an increase of water storage in forested lands as compared with the open prairie land adjoining.

In considering the effect of forest cover on the catchment of precipitation, the influence of the trees themselves in intercepting both rainfall and snowfall must be considered. This influence is particularly important in light rains when a large proportion of the precipitation may be intercepted by the

* *Monthly Weather Review*, Supplement No. 17.

† "Run-Off from Small Agricultural Areas," *Journal of Agricultural Research*, Vol. 34, No. 9.

‡ *Swiss Forestry Journal*, March, 1927.

§ "A Preliminary Statement of the White Mountains of New Hampshire," by George N. Smith, U. S. Geological Survey Rept., No. 13.

‖ "The Relation of the Smaller Forest Areas in Non-Forested Regions to Evaporation and Movement of Soil Water," Iowa Academy of Science, Vol. XXVII, 1920, pp. 137–157.

tree crowns and re-evaporated into the air without having reached the forest soil.

Robert E. Horton, M. Am. Soc. C. E., has discussed this subject,* and has also investigated the loss of water by transpiration from trees.† He indicates that from 0.02 to 0.07 in. of rainfall per shower may be intercepted, and that the total loss for the summer season may be as high as 40% of the total precipitation.

Mr. H. S. Graves‡ has reported loss through interception in forests as varying from 20 to 60% of precipitation. While no very exact information exists, and the interception is admittedly less in regions of heavy rains, the loss through interception of the crowns of trees must be considered, together with transpiration and ground storage, as a factor in flood control.

It is evident that there is still a large field for research in determining the exact relation of the forest to the retention of flood waters. In spite of this fact, the evidence collected thus far is of a positive nature and indicates a material and beneficial influence.

The rapidly increasing number of storage reservoirs for power purposes and the increasing value of forest-covered water-sheds as sources of potable water for neighboring cities, are additional reasons for the maintenance of good forest conditions on the upper reaches of mountain streams. Not only should the abandoned fields of the Appalachian Mountains and Cumberland Plateau tributary to the Ohio drainage be forested, but many areas of cleared land in the hill sections of Illinois, Indiana, and Ohio, can be restored to forest cover both with benefit to the flood situation and profit to the owners.

In summarizing the influence of the forest on flood control, the outstanding service which it accomplishes is the protection of the rougher land from erosion. This, in turn, has a direct influence through the control of silting on practically all the engineering work undertaken.

The forest is a natural storage for water, because it delays run-off, intercepts rainfall, and retains the fine soil and humus in place on the steeper slopes.

APPENDIX

China has long been held up as a terrible example of forest destruction. Mr. W. C. Lowdermilk§ has recently prepared a preliminary report on the problem of forest conservation in Shansi, China. The following abstract is taken from this report:

"The Taiyuan plain was suffering from a long drought when we reached Taiyuanfu. The crops were dying for want of rain, even the wild grass had not started (July 3) and a famine was feared.

"The rain clouds broke shortly after our arrival. The moisture which had been accumulating in the atmosphere, superheated by the roasting hills, came down in torrential volumes. The streams were suddenly overcharged with raging waters, heavily laden with mud and silt. The water supply so sorely

* "Rainfall Interception," *Monthly Weather Review*, Vol. 47, No. 9, pp. 603–623.

† "Transpiration by Forest Trees," *Monthly Weather Review*, Vol. 51, No. 11, pp. 571–581.

‡ *Monthly Weather Review*, Vol. 42, December, 1914, p. 671.

§ "The Problem of Forest Conservation in Shansi, China." (In mss.)

needed for agricultural crops quickly ran off the steep barren slopes and brought floods and destruction in the place of the desired benefits of rain after drought. Despite the efforts made to catch the flood waters for irrigation, most of them soon passed by and were gone. Dry weather in the fall months again made the conditions of the people serious and threatened them with famine. Being deprived of their soil and forest cover, the mountains no longer serve as a storehouse of water to be made available, when most sorely needed, to the fields in the plains.

"Our party was able to study these forces at work and to follow the processes of the denudation of mountain slopes. Far back at the foot of the famous Lu Yah Shan near the Great Wall was found a small remnant of what was once a great forest cover. Now only a few score of square li of the original forest are left. Yet here may be found the beneficent influences of a forest cover. From out this natural forest area, consisting of trees and a deep porous soil of decaying vegetable matter, flows now a beautiful perennial stream. It was scarcely affected by the heavy rains of the summer [1924], whereas the stream beds draining the barren slopes became raging torrents after an hour of rainfall. But now this last remnant of true and natural forest is being attacked, for not less than 500 wood cutters began in the summer of 1924 to clear the tract.

"The cutting of trees is not the cause of forest destruction in the mountains of Shansi; it is, on the contrary, the digging up of the soil on the mountain slopes for agricultural crops that begins the process of destroying the soil cover."

The following is an abstract from the paper by Mr. H. Burger* previously mentioned:

In two small water-sheds in the Canton of Berne, Switzerland, measurements have been taken which indicate very definitely the effect of the forest cover on run-off. These two water-sheds are similar in altitude, exposure, slope, and character of soil. They are 3 km. distant from each other. One (the Sperbelgraben) is entirely wooded, the other (the Rappengraben) is only 35% wooded. The entirely wooded basin has an area of 56 hectares, the other, 70 hectares.

On June 22, 1926, at 5:00 P. M., a violent storm took place, during the course of which 28 mm. of rain fell on the wooded basin and 29 mm. on the other. The maximum flow from the partly forested basin was 1 079 liters per sec. for each 100 hectares and the crest occurred ½ hour after the beginning of the rain. The maximum flow from the completely forested basin was only 240 liters per sec. and the crest occurred about 5 hours after the storm began.

The total run-off per square kilometer during the period from the beginning of the storm until June 23 at 6:00 A. M., was 7 696 cu. m. on the partly forested basin and only 4 720 cu. m. on the completely forested basin. By subtracting the amount of normal flow, observed just before the storm in each case, the run-off due entirely to the storm is obtained. This is 6 184 cu. m. for the partly forested basin and 2 970 cu. m. for the completely forested basin. On the partly forested basin, 21% of the storm-water ran off; on the completely forested area only 10 per cent.

The purpose of the experiments made by C. E. Ramser, M. Am. Soc. C. E., was to determine the rates of run-off from small agricultural areas. They consisted in making rainfall and run-off measurements on six water-sheds ranging in area from 1¼ acres to 112 acres. The following quotation from Mr. Ramser's report† indicates the effect of timber upon rate of run-off:

* *Swiss Forestry Journal*, March, 1927.

† "Run-Off from Small Agricultural Areas." *Journal of Agricultural Research*, Vol. 34, No. 9, pp. 797–823.

"The effect of timber upon the rate of run-off is shown by a comparison of the results obtained for Water-Sheds Nos. 1 and 4, where the timbered areas were 14 and 38.9% of the respective water-sheds. In Table 8 it is seen that the run-off coefficients for Water-Shed No. 1, for all rains except that of July 18, range from 0.33 to 0.49, and for Water-Shed No. 4, from 0.22 to 0.35. These values show quite conclusively that timber has a decided influence in reducing the rate of run-off from a water-shed. However, the results obtained for the rain of July 18—for which run-off coefficients of 0.51 and 0.46 for Water-Sheds Nos. 1 and 4, respectively, were obtained—tend to show that the effect of timber in reducing run-off is slight when the maximum rate of run-

TABLE 8.—EFFECT OF TIMBER ON RUN-OFF COEFFICIENTS FOR WATER-SHEDS NOS. 1 AND 4; WATER-SHED NO. 1, 14.0 PER CENT. IN TIMBER; WATER-SHED NO. 4, 38.9 PER CENT. IN TIMBER.

Date of rain, 1918.	AVERAGE RATE OF RAINFALL DURING TIME OF CONCENTRATION, IN INCHES PER HOUR.		COEFFICIENT OF RUN-OFF (RATIO OF MAXIMUM RATE OF RUN-OFF OF AVERAGE RATE OF RAINFALL).		RAINFALL PRIOR TO PERIOD TAKEN AS TIME OF CONCENTRATION, IN INCHES.	
	Water-shed No. 1.	Water-shed No. 4.	Water-shed No. 1.	Water-shed No. 4.	Water-shed No. 1.	Water-shed No. 4.
February 19	3.40	1.89	0.40	0.22	0.39	0.37
April 16	3.84	2.92	0.33	0.29	1.21	1.19
April 28	4.44	3.77	0.41	0.25	0.09	0.09
May 7	3.60	3.51	0.38	0.28	0.18	0.18
May 12	1.92	1.89	0.41	0.30	0.10	0.10
May 23	3.12	2.83	0.49	0.35	0.38	0.38
June 1	3.24	3.00	0.36	0.28	0.46	0.38
June 6, 11:00 P. M.	3.28	2.14	0.42	0.28	0.72	0.69
July 18	4.20	3.51	0.51	0.46	0.76	0.71

off occurs after considerable rain already has fallen. This is to be explained by the fact that interception and percolation on timbered areas are much greater at the beginning of a rain than later, so that an increasingly greater proportion of the rainfall runs off as the rain continues. The falling rain is intercepted by the trees, and the cover of leaves on the ground, until saturated, absorbs a large portion of the rainfall. On April 28 the average rates of rainfall were greater than on July 18, yet the rates and coefficients of run-off were smaller, being 0.41 and 0.25 as compared with 0.51 and 0.46 for the rains of April 28 and July 18, respectively. However, the rain that fell prior to the time of concentration on April 28 was 0.09 in. for both water-sheds, while on July 18, it was 0.76 in. for Water-Shed No. 1 and 0.71 in. for Water-Shed No. 4."

RESERVOIRS FOR MISSISSIPPI VALLEY FLOOD PROTECTION

By William Kelly,* M. Am. Soc. C. E.

For many years there has been an insistent demand for reservoirs to regulate stream flow throughout the United States in the interest of flood protection, navigation, irrigation, drainage, power development, and various other works connected with water resources. In fact, the idea that reservoirs will cure most of the evils from which the country is suffering has been widely accepted as more or less axiomatic in spite of the fact that such projects as have been studied in detail have presented so many difficulties that few of them have been found economically feasible. For five years, as Chief Engineer of the Federal Water Power Commission, the writer spent most of his time studying reservoir possibilities throughout the country to determine whether proposed power developments would conform to a general plan insuring maximum economic use of the water resources for all purposes, including irrigation, flood protection, navigation, etc. As a result of this experience the writer, while not accepting reservoirs as cure-alls, believed that they might form a valuable part of a complete Mississippi flood project.

So far as the Mississippi Valley is concerned, the Mississippi River Commission has often considered the subject of reservoirs, but has never found a possibility of obtaining results that would justify a recommendation for diverting any of its limited funds for this purpose. Many think that the reservoir plan for Mississippi flood relief has not received the consideration it merits. Be that as it may, General Jadwin, the Chief of Engineers, has determined to have the subject studied as thoroughly as all other phases of the flood problem and if there are any reservoirs that can profitably be developed for Mississippi flood protection, they will undoubtedly be included in the project recommended.

It should be noted that the policy of giving Federal aid for flood protection is a quite recent development. Federal jurisdiction over streams is conferred by the Commerce Clause of the Constitution. Twenty-five years ago such jurisdiction and responsibility was limited strictly to improvements for navigation. There has been no change in the Constitution in this respect, but gradually popular demand has caused Congress to extend Federal assistance for flood protection on the principal navigable streams to a point where its relation to navigation improvements is almost lost to sight. Perhaps if Congress had not so acted the public demand would have been strong enough to amend the Constitution.

The great Mississippi flood of 1927 has renewed the demand for further Federal activity, and in recognition of this demand and of the policies developed by Congress, the Administration has directed studies to be made of

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the Mississippi flood problem on a much more comprehensive basis than ever before. Early in June, 1927, a board of seven Engineer Officers was appointed to study the reservoir problem. The Board was given broad instructions to consider the question from every angle. It was authorized to call upon District Engineer Officers of the Army for any assistance needed, and to get data from every source where they could be found. The only limitation placed on the Board was that its report should be completed in time to be considered with the reports of boards studying other phases of the flood problem, so that any reservoirs found to be feasible, could be included in a complete report on Mississippi Valley flood protection to be submitted to Congress in December, 1927.

The Reservoir Board immediately laid out a program of studies along the following lines:

1.—To determine the effect on Mississippi floods of the larger reservoir systems already built in the Mississippi Drainage Basin as a practicable indication of what can be done.

2.—To determine reservoir capacities, costs, and effect on Mississippi floods of all practicable reservoirs on the tributaries of the Mississippi, both when operated primarily in the interest of Mississippi floods and when operated to get the maximum economic results, considering all uses of the reservoirs.

3.—To determine reservoir possibilities in the valley of the Mississippi and its main tributaries.

The task set the Board is a tremendous one to complete in the time available; but thanks to the excellent organization placed at its disposition and to the data available from previous studies, it is believed that the results obtained will be sufficiently complete and reliable to determine what can be done with reservoirs to ameliorate floods in the Mississippi, and what such amelioration will cost. Unfortunately, the final results of the Board's studies are not as yet available (October, 1927), so that this paper is no more than the writer's digest of the data now at hand. Some of the figures will doubtless be modified when the final report is made.

In order to determine the effect of reservoirs on Mississippi flood flows, it is necessary to establish some rule for reservoir operation. Practically all large Mississippi floods have occurred during the 90 days from February 15 to May 15. While there have been a few fair-sized floods both before and afterward, it is believed that this period fairly covers the time within which disastrous floods will occur.

The time consumed by water released from reservoirs on tributaries in traveling to the Mississippi varies from 3 days to 5 weeks, with an average of more than 2 weeks. Large Mississippi floods are produced by widespread heavy rainfall occurring throughout the valley when the river is already running full. The Board has been unable to find any means of predicting a large flood in the Mississippi as much as two weeks in advance of its occurrence. Hence, on the average the best dependable results from reservoirs will be obtained by filling them as uniformly as the stream flow will permit so as to hold water back from the Mississippi during the 90 days of probable flood,

February 15 to May 15. In its studies the Board has adopted such a procedure as the rule for reservoir operation. It is possible that for some of the reservoirs a more efficient rule of regulating might be found as a result of experience in their operation, but it is not likely that the increased efficiency would more than offset the probable error due to determining the dependable effect from only five floods, as the Board has been forced to do. A brief résumé of the studies on existing reservoir systems will be given.

Miami Conservancy District

These reservoirs are designed to protect the cities on the Miami River from disastrous local floods. Their action, which is automatic, is effective only during floods of considerable magnitude. It consists of storing all the flood flow in excess of the capacity of the valley below the reservoirs. As soon as the natural flow has fallen below the danger point, this stored water is discharged at a rapid rate. The effect on moderate local floods is a minor one and small floods are entirely unaffected. The costs and capacities of the reservoirs are as given in Table 16.

TABLE 16.—Costs and Capacities of Miami Conservancy District Reservoirs.

Reservoir.	Capacity, in acre-feet.	Cost.	Cost, per acre-foot.
Taylorsville	186 000	$4 170 000	$22.40
Lockington	70 000	2 020 000	28.90
Huffman	167 000	3 680 000	21.70
Englewood	312 000	4 050 000	13.00
Germantown	106 000	1 620 000	15.30
Total	841 000	$15 490 000	$18.40

These works appear to have been very successful in affording the designed protection to the Miami Valley. Their effect on the Ohio and Mississippi floods is to decrease the flood flow for 1 day, or 2 days, and then increase it by a somewhat smaller amount for several days. Whether the net result is to increase or decrease the maximum flow of the larger river is mainly a matter of chance.

This effect was computed for the years of the last five great Mississippi floods. Table 17 shows the results on the Ohio River.

TABLE 17.—Effect of Miami Reservoirs on Ohio River Floods.

Year.	Maximum Natural Discharge.		Maximum Discharge, with Reservoir Operating.		Effect of Miami reservoirs, in second-feet.
	Quantity, in second-feet.	Date.	Quantity, in second-feet.	Date.	
1912	558 000	March 30	553 000	March 30	5 000, decrease
1913	868 000	March 31	890 000	March 30	22 000, increase
1916	520 000	April 2	520 000	April 2	No effect
1922	540 000	March 19	540 000	March 19	No effect
1927	396 000	March 26	398 000	March 22	2 000, increase

It appears from Table 17, therefore, that these reservoirs have no dependable effect in reducing the magnitude of maximum floods of the Ohio River and that, in some instances, the effect may be adverse.

The effect on the floods of the Mississippi is dependent on the time required for the effects at the mouth of the Miami to be felt at Cairo, Ill. This has been taken to be nine days. If the effect of "valley storage" in the Ohio between the Miami and Cairo be neglected, the results are as given in Table 18.

TABLE 18.—Effect of Miami Reservoirs on Mississippi River Floods.

Year.	Maximum Natural Discharge.		Maximum Natural Discharge, with Miami Reservoirs Operating.		Difference, in second-feet.
	Quantity, in second-feet.	Date.	Quantity, in second-feet.	Date.	
1912	1 946 000	April 5	1 945 000	April 5	1 000, decrease
1913	1 677 000	April 4	1 725 000	April 7	48 000, increase
1916	1 903 000	February 5	1 903 000	February 5	No effect
1922*	1 582 000	April 25	1 584 000	April 25	2 000, increase
1922*	1 595 000	March 25	1 596 000	March 25	1 000, increase
1927	1 800 000	April 19	1 800 000	April 19	No effect

* There were two flood peaks of nearly equal magnitude at Cairo in 1922.

Table 18 shows that the effect on the Mississippi may be either beneficial or the reverse. The effect of the "valley storage" on variations of flow at the mouth of the Miami of only a few days' duration is such that the actual results at Cairo would be similar to those in Table 18, but of less magnitude.

If these Miami reservoirs, instead of being used as detention basins for the protection of that valley, had been used as reservoirs for the protection of the Mississippi under the operating schedule adopted by the Reservoir Board, their effect would have been to reduce the crest at Cairo by quantities varying between 1 000 and 7 000 sec-ft. for different flood-years. The dependable effect would not exceed 0.2 in. on the Cairo gauge. If the reservoirs were used in this manner a new method of protection for the Miami Valley would have to be provided, and the total cost probably would be in excess of $30 000 000.

Keokuk Reservoir

As now operated, the Keokuk Reservoir has no measurable effect on Mississippi floods. If operation were modified in the interests of flood protection the pond might be drawn down 6 ft. below the normal stage by February 5, and the resulting 142 800 acre-ft. filled uniformly over the probable 90-day flood period. This would give a reduction of 790 sec-ft. on Mississippi floods, and would lower the flood height at Cairo by less than $\frac{1}{7}$ in. It would entail a power loss of 16 000 000 kw-hr., the value of which, if replaced by steam power, is greater than the cost of obtaining similar flood benefit by other means.

The level of the pool cannot be raised more than 3 ft. without excessive flowage damages. It might be raised 3 ft., giving 100 000 acre-ft. additional storage, at an estimated cost of about $5 200 000, or $52 per acre-ft. Such a

storage would reduce the flood flow about 560 sec-ft. and the crest height at Cairo about 0.1 in. The cost would be more than ten times as much as the cost of levees to take care of the same flow at the top of the flood.

Reclamation Service Reservoirs

The U. S. Reclamation Service has built a number of large irrigation reservoirs in the Mississippi Drainage Basin. To determine what effect they have had on Mississippi floods, data are available for eight of these reservoirs for 1922 and 1927, as shown in Table 19.

TABLE 19.—Irrigation Reservoirs in Mississippi Drainage Basin.

Name.	Capacity, in acre-feet.	Approximate cost.	Cost, per acre-foot.
Pathfinder	1 020 000	$2 000 000	$ 2
Lake Mintare	60 800	500 000	8
Lake Alice	12 000	100 000	8
Winter Creek	3 700	100 000	27
Belle Fourche	208 000	1 200 000	6
Willow Creek	17 000	200 000	12
Shoshone	457 000	1 400 000	3
Nelson	40 000	200 000	5
Total	1 813 500	$5 700 000	$3.15 (average)

At the time when the water that was to contribute to the crest of the 1927 flood was passing these sites the Shoshone Reservoir was discharging 550 sec-ft. more than normally. The Pathfinder and Belle Fourche Dams were storing at moderate rates, about 600 and 250 sec-ft., respectively. At the other sites, stream flow and storage were both very low. It should be noted that the time in question comes in March, before the first spring thaws occur in this mountainous northern country. The aggregate storage rate for the eight reservoirs is estimated at 325 sec-ft.

In 1922 the flood had two practically equal crests at Cairo; one late in March and one late in April. From the earlier one the irrigation reservoirs held back about 900 sec-ft., while from the later they withheld more than 3 200 sec-ft.

The quantity held back in the 1927 flood was equivalent to about $\frac{1}{3}$ in. on the gauge at Cairo. This is based on the actual operation of the irrigation reservoirs. With the exception of Shoshone, it appears that they were storing practically all the water there was and that the volume could not have been increased materially. The Shoshone Reservoir was being used for the joint purpose of irrigation and power development. Had it been operated primarily for Mississippi flood control, it could probably have stored about 500 sec-ft.

Reservoirs at Head-Waters of the Mississippi

In the State of Minnesota, on the head-waters of the Mississippi River, six reservoirs are maintained by the United States. They are operated pri-

marily for the purpose of increasing the navigable depths in the Upper Mississippi above Lake Pepin during periods of low water. They also serve to increase the amount of water power developed during these periods and to diminish the damage caused by local floods. These purposes are accomplished in a fairly satisfactory manner; but ever since the reservoirs were finished the War Department has been receiving frequent demands to operate them in some other way to benefit other interests.

The cost of these reservoirs was very small as they were formed by the construction of low dams at the outlets of natural lakes at a time when price levels were down and land and flowage rights were cheap, as the country was largely undeveloped. The District Engineer at St. Paul reports the capacities and costs as in Table 20.

TABLE 20.—HEAD-WATER RESERVOIRS, MISSISSIPPI RIVER.

Name.	Capacity, in acre-feet.	Cost.	Cost per acre-foot.
Lake Winnibigoshish	967 170	$368 200	$0.38
Leech Lake	743 840	246 800	0.33
Lake Pokegama	120 760	197 000	1.63
Sandy Lake	72 500	218 000	3.00
Pine River Reservoir	177 520	215 500	1.21
Gull Lake	70 970	77 200	1.08
Total	2 152 226	$1 322 700	$0.61

The effect of the operation of these reservoirs in reducing the floods of the Lower Mississippi was studied for each of the great floods of recent years with the following results.

Flood of 1912.—In this year the flood crest reached Cairo in the early part of April and the operation of the Minnesota Reservoir System caused a reduction of about 1 100 sec-ft., or less than 0.2 in. on the gauge at Cairo.

Flood of 1913.—The flood crest reached Cairo early in April. The Upper Mississippi was at a low stage at the time corresponding to this flood wave, and the reservoirs were discharging water in order to maintain navigable depths. This resulted in increasing (rather than decreasing) the flood discharge by about 230 sec-ft.

Flood of 1916.—The flood crest reached Cairo early in February. Conditions in the Upper Mississippi were similar to those in 1913 and the operation of the reservoirs increased the flood discharge about 250 sec-ft.

Flood of 1922.—This flood had two approximately equal crests at Cairo, the first late in March and the second late in April. The operation of the reservoirs reduced the earlier crest by 500 sec-ft., and the later crest by 1 700 sec-ft., or about $\frac{1}{3}$ in. on the gauge at Cairo.

Flood of 1927.—The flood crest reached Cairo just after the middle of April. Operation of the reservoirs caused a reduction of about 1 200 sec-ft., or slightly more than 0.2 in. on the gauge at Cairo.

In 1912, 1922, and 1927, the reservoirs were storing the entire flow of their streams except the minimum that had to be passed to prevent damage to

navigation and power development. If they had been operated solely for the benefit of the Mississippi floods they could have done no more without damage to these interests. In 1913 and 1916, they were contributing a small increase to the Mississippi floods to the benefit of navigation and power.

Summary.—Leaving out the Miami regulation which is more apt to be detrimental than beneficial, the three projects just discussed have a combined storage capacity of about 4 150 000 acre-ft. As now operated their combined dependable reduction of Mississippi flood peaks is about 75 sec-ft.; that is, to reduce past floods 1 sec.-ft. has required more than 55 300 acre-ft. of storage. If these reservoirs had been operated primarily for Mississippi flood protection they might have reduced Mississippi floods by 2 200 sec-ft., or nearly thirty times as much. These ratios are interesting, but should not be used indiscriminately.

Possible Reservoirs on the Tributaries of the Mississippi River

For this next study the water-shed of the Mississippi was divided into five principal divisions, as follows: (1) The Ohio River and its tributaries; (2) the Upper Mississippi River; (3) the Missouri River; (4) the Arkansas River group of tributaries; and (5) the Red River group of tributaries.

The Upper Mississippi has been taken to include that part of the river between St. Louis, Mo., and Cairo. The Arkansas group includes the White, Black, and St. Francis Rivers and minor tributaries between Cairo and Arkansas City, Ark. The Red River group includes the Ouachita and Yazoo Rivers and neighboring small streams. Each of these divisions differs from the others in its run-off characteristics and also in its effect on Mississippi floods. Table 21 shows the drainage area of each division and the percentage of flood flow contributed by each in recent floods.

TABLE 21.—Principal Tributaries of Mississippi River with Drainage Areas and Percentage of Flows Contributed.

Division.	Area, in square miles.	Percentage of Flow at Cairo, Ill.					
		1912.	1913.	1916.	1922*.	1927.	Average.
Ohio	203 000	61	72	57	70 57	46	60
Upper Mississippi	186 000	23	25	32	19 28	26	28
Missouri	530 000	16	3	11	11 15	18	12
		Percentage of Flow at the Mouth of the Arkansas River.					
Arkansas	202 000	12	9	17	19	40	19
		Percentage of Flow at the Mouth of the Red River.					
Red	104 000	13	8	13	15	14	13

* In 1922 there were two nearly equal flood crests at Cairo.

The percentages of flow at Cairo are fairly accurate, but those at the Arkansas and Red Rivers are rather uncertain, due to difficulties in determining the actual flow in the Mississippi at these points.

The District Officers reported on all reservoirs that could be found with a capacity of 100 000 acre-ft. or more. In all, 321 reservoirs were included in the reports. Of these, 124 were thrown out as impracticable, or unnecessary, chiefly because there was not sufficient water to fill them at the time they are needed.

Table 22 gives a summary of the 197 reservoirs which have been studied in detail.

TABLE 22.—Summary of Possible Flood Control Reservoirs.

Division.	Number of reservoirs.	Total capacity, in acre-feet.	Total cost.	Average cost per acre-foot.	Percentage of drainage area above lowest reservoir.	Percentage of total rainfall of Mississippi River above lowest reservoir.
Ohio	90	29 200 000	$543 000 000	$18.60	30	Not available.
Upper Mississippi	54	17 400 000	305 000 000	17.50	86	
Missouri	7	32 700 000	232 000 000	7.10	84	
Arkansas	32	23 500 000	201 000 000	8.60	58	
Red	14	8 700 000	32 000 000	3.70	45	
Total	197	111 500 000	$1 313 000 000	$11.80	65	45 in 1927

From the hydrographs and storage curves of the reservoirs, graphs were prepared showing the aggregate storage rate in the reservoirs of each of the five divisions for each day of the 90-day storage period. The actual reduction of flow in the Mississippi Valley is quite different from the sum of the rates of storage at the various reservoirs, because of the retarding and equalizing effect of the storage in the valleys of the tributaries between the reservoirs and the Mississippi. For example, between high and low water the Ohio has a storage capacity in the main valley of about 12 000 000 acre-ft., more than 40% of the capacity of all the reservoirs available on its water-shed.

Computations to determine the effect of this valley storage are not yet completed (October, 1927). A rough approximation, therefore, has been used to obtain the figures in the computations to follow. The reduction in maximum flood discharge that would have been brought about by the reservoirs of each division is shown approximately in Table 23. In this table, as in the previous tables, the results at Cairo are fair, but those at points below are not so good, owing to doubt as to the flow in the Mississippi River at these lower points.

The effect on gauge heights of this reduction depends on the increment of discharge corresponding to a change of stage of 1 ft. near the crest of high confined floods. The increment of discharge varies considerably in

different floods due principally to changes in the slope of the river caused by variations in relative contributions by the tributaries. Studies on this subject are still in progress, but it is believed that the following discharges can be taken as fairly safe: (*a*) 65 000 sec-ft. at Cairo; (*b*) 80 000 sec-ft. at the mouth of the Arkansas River; and (*c*) 75 000 sec-ft. at the mouth of the Red River. The last quantity is a joint increment for the Lower Mississippi and Atchafalaya Rivers.

TABLE 23.—NET MAXIMUM FLOOD DISCHARGE AS REDUCED BY RESERVOIRS, IN SECOND-FEET.

Division.	1912.	1913.	1922.	1927.	Minimum.
Ohio	195 000	210 000	200 000	140 000	140 000
Upper Mississippi	80 000	85 000	80 000	95 000	80 000
Missouri	140 000	90 000	90 000	110 000	90 000
Total at Cairo	415 000	385 000	370 000	345 000	345 000
Arkansas Group	120 000	45 000	130 000	250 000	45 000
Total at mouth of Arkansas	600 000	340 000	500 000	600 000	340 000
Red River Group	20 000	8 000	35 000	50 000	8 000
Total at mouth of Red River	430 000	250 000	420 000	450 000	250 000

Table 24 shows the approximate dependable effect reservoirs would have had in reducing maximum flood heights that have heretofore occurred.

TABLE 24.—EFFECT OF RESERVOIRS IN REDUCING MAXIMUM FLOOD HEIGHTS.*

Division.	At Cairo, in feet.	At mouth of Arkansas River, in feet.	At mouth of Red River, in feet.
Ohio	2.15	1.75	1.88
Upper Mississippi	1.23	1.00	1.08
Missouri	1.88	1.12	1.21
Total at Cairo	5.30	....	....
Arkansas Group	....	0.56	0.61
Total at mouth of Arkansas River	....	4.25	....
Red River Group	....	....	0.11
Total at mouth of Red River	....	....	3.36

* The total effect of several divisions on the same gauge is not the same as the sum of their effects on that gauge.

The estimated cost of reservoirs to produce this lowering of gauge heights is $1 313 000 000.

In Table 25 is shown the cost of reducing the Mississippi River discharge 1 sec-ft. by reservoirs in each of the five divisions.

The studies of other boards working on the Mississippi flood protection problem indicate that it is feasible to protect the valley against the maximum probable flood by levees, spillways, and by-passes, which means that reservoirs are not essential to a Mississippi flood project and, consequently, their adoption or rejection should rest on their economic value. The studies have not reached a point where final results can be given; but an estimate has been made of the difference in cost of protecting the valley by levees against the

maximum probable flood and against a flood of 300 000 sec-ft. less. The results are probably higher than the cost of any alternate method that might be adopted, such as the provision of a by-pass through Tensas Basin, or of a large amount of storage in St. Francis Basin. The results are given in Table 26.

TABLE 25.—COST OF REDUCING MISSISSIPPI RIVER DISCHARGE 1 SECOND-FOOT BY RESERVOIRS.*

Division.	Reservoir capacity, in acre-feet.	Cost of reservoirs.	Dependable reduction, in second-feet.	Cost per second-foot of reduction.
Ohio	29 200 000	$543 000 000	140 000	$3 870
Upper Mississippi	17 400 000	305 000 000	80 000	3 800
Missouri	32 700 000	232 000 000	90 000	2 590
Arkansas Group	23 500 000	201 000 000	45 000	4 490
Red River Group	8 700 000	32 000 000	8 000	4 020

* Reservoirs in the Arkansas and Red River groups reduce the flow only in that part of the Mississippi River lying below them, respectively.

From Table 26 it appears that a reduction of flow at Cairo by 1 sec-ft., is worth $860. The cost of obtaining it by reservoirs on the Ohio would be $3 870, or 4.5 times as much; on the Upper Mississippi, $3 800, or 4.4 times as much; and on the Missouri, $2 590, or 3 times as much.

TABLE 26.—SAVING IN COST OF FLOOD PROTECTION BY LEVEES DUE TO A REDUCTION OF 300 000 SECOND-FEET IN MAXIMUM FLOOD FLOW.

Part of river.	Estimated cost.	Cost per second-foot of reduction.
Cairo to mouth of Arkansas	$ 27 000 000	$ 90.00
Mouth of Arkansas River to mouth of Red River	192 000 000	640.00
Mouth of Red River to Gulf	39 000 000	130.00
Total	$258 000 000	$860.00

A reduction in the discharge of the Arkansas group of 1 sec-ft., is worth $640 plus $130, or $770. The cost of obtaining this by reservoirs is $4 490, or 5.8 times as much. A similar reduction on the Red River group is worth only $130 and the cost of obtaining it by reservoirs is $4 020, or 31 times as much.

This relation can also be set up by comparing the cost of an acre-foot of storage with the cost of an equivalent amount of protection by levees. Table 27 shows such a comparison.

Columns (5) and (6), Table 27, show the costs, which should be compared with the values in Column (4). It appears that only in the Upper Mississippi Division are any reservoirs found the cost of which is less than their computed values for flood protection. Four such reservoirs are found in that Division. Their combined capacities total about 846 000 acre-ft.; and they would lower

the gauge height at Cairo about ¾ in. A further analysis of them is being made to see what would be the effect on other interests of taking these reservoirs for Mississippi flood protection.

TABLE 27.—COMPARISON OF COST OF AN ACRE-FOOT OF STORAGE TO COST OF EQUIVALENT LEVEE PROTECTION.

Division. (1)	Value of 1 sec-ft. (2)	Acre-feet required for 1 sec-ft. (3)	Value of 1 acre-ft. (4)	Average cost of 1 acre-ft. (5)	Cost of 1 acre-ft. in cheapest reservoir. (6)
Ohio	$860	208	$4.13	$18.60	$6.00
Upper Mississippi	860	217	3.96	17.50	2.50
Missouri	860	364	2.36	7.10	4.22
Arkansas Group	770	522	1.48	8.60	3.94
Red Group	130	1 087	0.12	3.70	1.97

The studies of the Board have shown that many reservoir sites have an economic value for local use that is greater than their value for Mississippi flood protection, so that before any reservoirs are appropriated exclusively for such protection, a careful analysis should be made to ascertain what the effect of this action would be on local interests. On the assumption that the best interests of the country will be served only when reservoirs are used to produce their maximum economic value, it may be concluded that if reservoirs have a large potential local value they should not be lightly appropriated for exclusive Mississippi flood use. It has been found that operation of reservoirs for local purposes will generally greatly reduce their value for Mississippi flood protection, and, in fact, may even make that value a negative quantity, as already shown in the case of the Miami Conservancy Reservoirs and the reservoirs at the head of the Mississippi.

The costs of reservoirs noted in this paper are the total costs as estimated by the District Engineers. In some cases benefits would result to other interests, which might somewhat reduce the costs to be charged to Mississippi flood protection. The estimates of such benefits are not yet complete, but the indications are that, while they may justify further analysis of a few reservoirs, they will not materially change the general results.

RESERVOIRS FOR COMBINED USE

In addition to the studies of reservoirs to be built and operated primarily for Mississippi flood protection, the Board has made extensive studies of the results to be obtained should reservoirs be built and operated for their greatest economic value, considering all uses. Obviously, the time and funds available were not sufficient for completing such studies for all the streams in the Mississippi Drainage Basin. There are almost an infinite number of ways in which any stream can be developed and operated. The Board, therefore, concentrated its efforts on the streams on which other interests had proposed

and studied developments. The results of its studies are not yet (October, 1927), in such shape that a complete digest of them can be presented; but some of the results can be indicated.

In general, it appears that the value of reservoirs for local flood protection, irrigation, power, or navigation, or for a combination of such uses, is much greater than their value for Mississippi flood protection. In other words, Mississippi flood protection, if proper weight be given to economics, will generally be a minority interest in reservoir development, and the practical question arises as to how far a minority interest can go toward securing participation of the majority interests. Such an undertaking can be accomplished in only two ways: Either the minority interest must wait till the majority interest is ready to proceed, or it must undertake the financing for both interests and assume the risk of being able to dispose profitably of the majority interest after development is made.

There may be some streams on which the majority interests will be ready to proceed before the Mississippi flood project is carried out, and on such streams it may be practicable to secure a certain amount of Mississippi flood protection by contribution commensurate with its value. On the other hand, the need for such protection is immediate, and its cost, at best, promises to burden the Treasury with all or more than it should properly bear. There will doubtless be advocates ready, as in the past, to use the urge for Mississippi flood protection as a means to induce the Federal Government to undertake comprehensive projects for complete development of some or all of the streams, regardless of what proportion of the benefits may accrue to flood protection. Such projects should be studied and treated on their own merits, with due consideration to all interests; and it is to be hoped they will not be permitted to delay action on a Mississippi flood project should it not include them.

Thus far, the Board has found no reservoir projects that it feels justified in recommending for combined development, but, as stated previously, it recognizes that such projects may develop before the Mississippi flood project is too far completed to make use of them. At such time, it will not be difficult to reach a conclusion as to what use can be made of them.

Otherwise than those mentioned, it is believed that dependable benefits to Mississippi floods that may result from the development of reservoirs for combined uses, will not be large and that they should be counted on solely as contributing to the factor of safety on whatever Mississippi flood project is adopted. The Board is endeavoring to arrive at a measure of the probable benefits to Mississippi flood protection should the maximum economic development of all streams in the drainage basin be accomplished. At best, such an estimate cannot be accurate and perhaps the nearest approach to an answer will come from determining the effect on Mississippi floods of equating the flow of the tributaries as nearly as the available reservoir capacities will permit. From the studies to date (1927), it appears that it may not exceed 10% of the effect of the same reservoirs used primarily for Mississippi flood protection.

Valley Reservoirs

In addition to reservoirs on tributaries, the Board was charged with the duty of determining whether any feasible reservoirs could be found in the Valleys of the Mississippi and Ohio themselves.

The results of the studies on this subject are not yet completely available, but it appears that reservoirs could be built on the main river at Commerce, Mo., in the St. Francis Basin, at the mouth of the Yazoo, and in a few other places. No practicable reservoir sites were found in the main Valley of the Ohio.

The cost of the reservoir at Commerce, Mo., appears to be too great to make it feasible at this time, even when credited with all possible benefits from power navigation. The reservoirs at the mouths of the lower tributaries, such as the Yazoo, do not offer much promise. The St. Francis Basin offers the best reservoir possibilities in the valley. To create a reservoir there, however, will drown out several towns and necessitate the relocation of many miles of railroad and highway. The estimates of the flowage damages are not yet complete, and until available, no opinion as to the feasibility of the project is justified.

To conclude, the prospects for reservoirs as a part of a Mississippi flood project, are not very good. One after another of the possibilities have "washed out" till practically nothing is left, except certain relatively small reservoirs on the Upper Mississippi and certain reservoirs in the Mississippi Valley proper or at the mouths of the principal tributaries and even they do not look very promising.

The Board, urged on by the Chief of Engineers, has used every effort to find feasible reservoirs. Its studies have been far-reaching and while the limited time available has made it necessary to adopt certain "short cuts" and approximations, it is the writer's belief that the results will be sufficiently accurate to permit a dependable determination of the extent to which reservoirs may enter into the Mississippi flood project to be adopted.

THE BASIS OF THE CASE AGAINST RESERVOIRS FOR MISSISSIPPI FLOOD CONTROL

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There has been a fifty-year controversy between the proponents of reservoir control and of levee control for Mississippi floods. The writer is not attempting to settle that issue. He does not know the aggregate possibilities for reservoir control, and any estimate made without very extensive and thoroughgoing investigation can be but a guess.

It is a fundamental axiom of sound engineering that when great issues are at stake, no conclusion of large importance should be left to chance. Actual and definite studies of actual cases are necessary. Until they have been carried to a point where, as nearly as is reasonably possible, all major possibilities have been conclusively determined, there is no adequate basis for far-reaching engineering policy.

If, as is contended by the members of the Mississippi River Commission, the case against reservoir control for the Mississippi has been properly established, it is because a definite study of the matter has been made, and because a conclusion has been reached in accordance with the evidence. The transmission of an authoritative dogma from one generation to another will not suffice. Somewhere along the way that authoritative doctrine must have been established by scientific methods, and those methods and results must be subject to re-examination.

The writer decided that the best contribution he could make to this discussion was to try to discover the exact nature of the evidence which has led the Mississippi River Commission to reject reservoirs. He has read a vast amount of literature on the subject, especially the writings of members of the Commission and of Army Engineers, and he has constantly watched for references to other and authoritative opinions and reports.

The whole controversy has been clouded by two factors. First, it has been made to appear as an issue of levees *versus* reservoirs, on the assumption that they are mutually exclusive.

Such is not the case. In a great problem like that of the control of the Mississippi it would be almost miraculous if 100% of the desired results could be achieved by only one method. The real problem is to discover and appraise all possibilities, and then, by careful analysis and synthesis, to design a total improvement in which each method is introduced to whatever degree is most effective and economical. If even 10% of the desirable results can be secured by each of two or three other methods, then control by levees may be reduced to far more manageable limits.

So far as the writer can discover, up to 1927 this attitude of endeavoring to work out a solution that will use all desirable methods never has been taken by the Mississippi River Commission, but rather the attitude of an armed camp in the battle of levees and revetment against all other methods of control.

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Second, great confusion has been brought into the controversy by the absence of a basis for comparison. There frequently has been a general admission that reservoir control would be preferable to levee control if it were economically as feasible. To determine that levee control is less expensive it is necessary to know approximately what each would cost. Now, neither of these costs ever has been adequately determined. The Mississippi River Commission estimates of the discharges to be provided for by a levee system, and of the works ultimately necessary, always have been made by arbitrary "practical" methods, which by all the laws of probability were bound to fail. The writer agrees with the statement of N. C. Grover, M. Am. Soc. C. E., that students of hydrology have long known that a flood of the magnitude of that of 1927 was inevitable.* The failure to recognize flood possibilities is one of the causes of inadequate estimates.

That the official estimates have been presented to the public as the full cost of levees in a program of complete protection, there can be no doubt. The report of the Special Engineering Board of 1874-75, which led to the establishment of the Mississippi River Commission, largely established the policies by which it has been controlled. This report indicated a cost of less than $46 000 000 for complete control by levees.

The late H. M. Chittenden, M. Am. Soc. C. E., at one time Chief of Engineers U. S. Army, formerly was a recognized authority of the Army Engineers against reservoirs. In deciding against reservoir control on the basis of cost, he referred to the report of the Mississippi River Commission for 1896, as authority for the statement: "Take $40 000 000 and reinforce the entire levee system of the Mississippi. That will make it impregnable. * * *"†

In 1896 the levee system was only well begun. If $40 000 000 would make it impregnable, then it was almost certain that reservoir control would be more expensive.

In the 1896 report of the Chief of Engineers, U. S. Army, the estimated cost of complete protection by levees is $18 000 000. In 1912, the President of the Commission stated that $73 000 000 was ample to do all remaining necessary work.‡ In 1922, a member of the Commission stated:§

"There is no uncertainty as to the outlay that will be necessary to * * * complete flood control * * * neither is there material uncertainty as to the grade line to which the levees must be constructed * * * no increase in height is now contemplated, except to meet local conditions such as are due to shifting of the channel through chutes or cut-offs."

In 1924 the Chief of Engineers stated‖ that 90 000 000 cu. yd. would complete the work. In 1925, in an article entitled, "Why the Mississippi River Commission Sticks to the Policy of Levees Only",¶ the President of the Commission stated:

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* See p. 694.

† *Transactions*, Am. Soc. C. E., Vol. LXII (1909), p. 304.

‡ *Engineering Record*, October 26, 1912, Vol. 66, p. 471.

§ *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 1479.

‖ *Loc. cit.*, Vol. LXXXVII (1924), p. 976.

¶ *Engineering News-Record*, Vol. 94, p. 513.

"There are now completed or nearing completion 1 824 miles of levee line on the Mississippi River, containing 429 000 000 yards of earth, and requiring about 76 000 000 yards to finish it. That would have been the end of the Commission's levee building but for the fact that Congress has recently extended the Commission's jurisdiction to the tributaries and outlets. * * *"

Year by year the Commission has reported the approach of the levee system to final completion. Under the heading "Effect of Improvement", the Commission formerly reported:*

"It may be stated that in a general way the improvement [of the Mississippi] is providing a safe and adequate channel for navigation and is now in condition to prevent the destructive effects of floods in all except the most extreme high waters. * * *"

For the past three years, beginning with the report of 1924, this qualifying clause has been omitted, and the statement reads: "* * * is now in condition to prevent the destructive effects of floods".†

The writer believes he has quoted enough to indicate that the estimates of the Mississippi River Commission have been inadequate. These estimates of the cost necessary to complete the levee system seldom have been more than 20% of the true cost, and sometimes less than 10 per cent. The writer believes that this extreme inadequacy of its estimates has rendered difficult, if not impossible, any dependable comparison between the cost of levee protection and reservoir protection.

On the other hand, until recently there has not been even a remote effort on the part of the Commission to discover and to estimate the cost of the most favorable opportunities for reservoir control. Thus, there is the remarkable situation of fifty years of insistence of the superiority of levees over reservoirs, without a thorough-going engineering analysis of the cost of either.

Perhaps, however, the members of the Mississippi River Commission are so thoroughly informed concerning the principles and facts of reservoir control that their informal judgment is sufficient. The writer has searched their writings of nearly half a century to find evidence of that clear understanding, but he finds it to be almost completely lacking. They have many times given their reasons for rejecting reservoirs, the more important of which will be mentioned.

In 1912, the President of the Mississippi River Commission stated:‡ "To control the flow of every stream in the Mississippi Valley by reservoirs is a pretty large job, even for the United States Government, but that is what the control of the Mississippi during floods by reservoirs signifies." This statement, of course, is incorrect. A similar impression is repeatedly given by referring to the vast drainage area of 1 250 000 sq. miles to be controlled. In fact, although the Ohio drains but one-third of the drainage area of the Mississippi above Cairo, Ill., and but 16% of the drainage area of the whole river, it delivers from 55 to 85% of the flood waters that pass Cairo. The control of that third of the water-shed above Cairo would go far toward furnishing all the control necessary at Cairo, and for a considerable distance

* Rept., Chf. of Engrs., U. S. Army, 1923, Vol. I, p. 1862.
† *Loc. cit.*, 1926, Vol. I, p. 1793.
‡ *Engineering Record*, Vol. 67, May 3, 1913, p. 506.

below. These men reiterate that the Ohio supplies only a third of the total annual run-off at Cairo, as if that statement were significant. The fact that it delivers 55 to 85% of the flow of great floods at Cairo is the important item.

In 1922, another member of the Mississippi River Commission stated* that "this shifting of the locus of great floods over such great areas prevents reservoir control." In illustration he mentions five floods as originating in the Ohio and the floods of 1844 and 1903 as originating in the Mississippi above Cairo. This statement is misleading, for both the floods he mentioned as coming from the Mississippi above Cairo, although they were the two record floods on the upper river before 1927, did not furnish enough water to be of importance below the mouth of the Ohio. The 1903 flood, which did such damage below Cairo, was not the summer flood from the Upper Mississippi, but a spring flood from the Ohio.

The public is repeatedly informed that the Commission and the Army Engineers know the value and limitations of reservoirs because they built and operate those on the Upper Mississippi in Minnesota.† These Minnesota reservoirs referred to by both friends and enemies of reservoirs as "the greatest system of reservoirs in the world", have practically no significance in the problem of flood control for the Mississippi, except to a minor degree within the State of Minnesota. They control an exceedingly small part of the total flow, and are too far removed from the lower river to have any appreciable effect on floods. What the Mississippi River Commission has learned about flood control by the operation of the Minnesota reservoirs is nothing of importance to the lower river.

This same President of the Commission makes a statement that is frequently repeated by the Army Engineers, and has been taken by them almost as an axiom. It is that, because of the lack of sites lower down, flood-control reservoirs must be limited to the head-waters of streams.‡ Flood-control reservoirs are best located low down on the streams, as nearly as possible just above the property they are to protect. Contrary to the frequently repeated statements of the members of the Mississippi River Commission, suitable sites low down on the main tributaries probably do exist.

Many engineers look back to the classic paper against reservoirs by Gros, Inspector-General of the Department of Bridges and Roads of France, in which the same assumption was made that dams must be limited to head-waters. This being almost the only European paper on the subject published in the American technical press until recent years, it is almost the only one referred to in their discussion.§

This paper, first published in 1881, was then reasonably sound in view of the existing limitations of dam design, although it is partisan. In those days French dams must be of cut-stone masonry, founded on solid rock. To-day,

* *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 1462.

† *Engineering Record*, Vol. 66, p. 470; *Engineering News-Record*, Vol. 94, p. 557.

‡ *Transactions*, Am. Soc. C. E., Vol. LXII (1909), p. 351; *Engineering Record*, Vol. 66, p. 470.

§ *Engineering News*, Vol. 25, March 14, 1891, p. 258

some of Gros' most serious objections are of small importance. The dams of the Miami Conservancy District would have been totally beyond his horizon. Engineering design has been revolutionized, and the valleys of large rivers often furnish safer and better sites for great dams than the head-waters.

A far more important French discussion of about the same period is that by Lechalas in "River Hydraulics", published in Paris in 1884,* although it, too, is partly obsolete.

The assumption that sites for flood-control dams can be found only on head-waters permeates the thinking and writing of the Army Engineers and members of the Mississippi River Commission, and largely destroys any value their studies might otherwise have had.

A President of the Mississippi River Commission, who served as Chairman of the Special Committee on Floods and Flood Prevention of the Society, states that all the reservoirs of a certain suggested system on the Upper Ohio would hold the total flood flow of the Mississippi for only one day, and that, as a flood may last forty days, it is implied that forty times as much reservoir capacity would be necessary.† This kind of reasoning is found throughout the discussions of members of the Mississippi River Commission. It represents one of the defects in the current reasoning on the subject. Even in the great flood of 1927, a holding back of 20% of the total flood flow probably would have been more than adequate. While the excess flow at the crest of the flood may be one-half or more of the total flow, the crest flow lasts for only a small portion of the total flood period. It should be recognized, however, that reservoir storage never can be perfectly timed to hold back the crest only, and that therefore reservoirs seldom operate with the highest theoretical efficiency.

Members of the Commission and Army Engineers repeatedly state that reservoirs on the Ohio System would soon fill with silt and be useless,‡ yet a careful study by Stabler, with reference to a proposed system of reservoirs on the Ohio, indicated that if all the silt above them were held, they would lose less than 10% of their capacity in 700 years.§ In flood-control retarding basins like those of the Miami Conservancy District, not more than one-third of the annual silt load is held in the basins, so that the life of a retarding basin on the Ohio would run into thousands of years before 10% of its capacity would be lost.

Another member of the Mississippi River Commission states,‖ "Nature has greatly simplified the problem of flood control by making the drainage of the flooded lands, in the larger basins, comparatively simple." Yet it is a fact that there are few more difficult problems of drainage and flood control than are found in these same basins. In the great St. Francis and Yazoo Basins there are four or more streams emerging from the hills, each of which has a maximum flood flow of 100 000 to 150 000 cu. ft. per sec., with an initial

* "Théorie Général des Réservoirs d'Emmagasinement des Crues" from "Hydraulique Fluviale," by Lechalas, p. 434, Baudry et Cie., Paris, 1884. The Dayton-Morgan Engineering Co. has a translation of this article, prepared by K. C. Grant, M. Am. Soc. C. E.

† *Engineering Record*, Vol. 66, p. 471.

‡ *Engineering News*, Vol. 60, p. 378; *Transactions*, Am. Soc. C. E., Vol. LXII (1909), p. 351.

§ *Loc. cit.*, p. 649, 1908.

‖ *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 1467.

channel capacity in the basin of less than 3% that amount. Control of these streams by reservoirs is feasible and probably is the only feasible method within reasonable cost. Such reservoirs would help solve the general Mississippi problem, and without them much of the land protected by levees will still be subject to destructive local flooding after the great river is controlled. In the writings of the Army Engineers and of the Mississippi River Commission for forty years one finds scarcely a hint of this great problem. It nearly always has been ignored in their general plans.

Repeatedly, the objection is made that reservoirs must not be built because of the valuable farm land that would be flooded. In all the discussion of the subject by members of the Commission one sees no inclination to offset this loss by the gain of a greater area of fertile land along the Mississippi between the river channel and the levees. In referring to the valuable storage along the river, a President of the Mississippi River Commission states that the levees are "several miles apart", the river channel being somewhat less than a mile wide; but while he denounces the waste of land by reservoirs, he does not mention this greater loss of fertile land along the river. Much of the reservoir area would be river bed or rough wooded hillside, whereas most of the land along the Lower Mississippi between the river channel and the levees is very fertile, sandy loam. In this item, which is repeatedly referred to as a controlling objection, the gain by reservoirs would wholly or largely offset the loss in the reservoir areas.

The Miami Conservancy Basins have been in operation eight seasons. All the bottom-land above the dams continues to be farmed, and only a small fraction of 1% of the crops planted in them has been lost by the operation of the dams. The lands increase in fertility through the deposit of silt in early spring floods, and steadily increase in productiveness. The sweeping generalizations of members of the Commission on this point are unsound.

Repeatedly, the members of the Commission and Army Engineers condemn reservoirs because of the danger of failure of dams. This objection does not run against flood control alone, but against water supply, irrigation, and power development as well. Well-built dams are a permanent part of present civilization.

The disorganization of highways, railroads, and towns is repeatedly given as a conclusive objection to flood-control dams. Until definite studies are made these indefinite misgivings have little value in determining policies.

For the plans of the Miami Conservancy District, storage space was needed on the Mad River. The best dam site available was across a flat valley nearly a mile wide, with gravel and sand a hundred feet deep below the surface. Two important trunk-line railroads crossed the dam site and traversed the length of the valley almost at water level, with a ruling grade of only 0.2 per cent. An electric line paralleled the two railroads, although with more tolerance as to grades, and an old established town of 1 200 people was in the proposed basin. Main-line and branch highways crossed the site from end to end. Yet it paid to overcome all these difficulties and to build a dam on that site in order to supply only one of the seven major elements of a local improvement.

When one comes to face the Mississippi River problem in its entirety it may be found desirable to retard flood flows by the construction of dams under even greater difficulties and on a far larger scale.

Unsanitary conditions above dams are continually emphasized. The Miami Conservancy dams have not caused such conditions, and the storage areas in the Adirondacks are among the chief playgrounds of the nation. Floods have far more unsanitary effects in the low areas of Ohio River towns, and these conditions might be relieved by reservoirs.

It has been contended by Army Engineers that reservoirs would have to be paid for by the National Government, whereas flood control must be paid for locally. Their own policy on this point now seems to have changed.*

The dams of the Miami Conservancy District are repeatedly referred to as indicating the cost of reservoir flood control for the Mississippi. To secure the local benefits desired these dams could be built only above Dayton and Hamilton, Ohio, and mostly in a region thickly settled and traversed by many railroads and highways. More than a hundred public utilities were interfered with on that project. The President of the Mississippi River Commission† assumes that the whole cost of the Miami System was for dams and retarding basins, whereas a very considerable part was for channel control and levees. The high cost of channel improvement through the cities on that project could be used as an argument against levees and revetment as well as the cost of storing water can be used against reservoirs. For flood control on the Mississippi a much greater range of choice would exist. Even on the Miami River, a similar amount of storage for flood control on the Mississippi probably could have been secured at a far lower cost per acre-foot, and there would be a large amount of storage available. The writer is of the opinion that vast storage capacity is available well down on the Ohio and its main tributaries at a much lower cost than that of the Miami Conservancy District. Storage in the Miami Conservancy project cost about $18.00 per acre-ft., as compared with $2.65 per acre-ft., as an average for U. S. Reclamation Service projects.

The Miami dams were built for local flood control, and serve that purpose primarily. If they had been in operation during the 1913 flood they would scarcely have affected the flood stage exactly at the mouth of the Miami; but, because of the nature of the flood crest, due to increments lower down the Ohio, they would have reduced the crest stage at and below Louisville perhaps a foot. If the same dams with the same storage had been designed for Ohio River flood control they would have lowered the flood stage on the Ohio 3 to 5 ft. in a flood like that of 1913. The Miami Conservancy dams are more massive and more safely built than is customary, and would be adequate for any type of operation, so far as their strength is concerned.

The writer came to have an intimate acquaintance with the late General Chittenden and feels free to refer to his writings because he has known but few engineers for whose intelligence and intellectual integrity he has had such high regard, and because General Chittenden was recognized as an authority on this subject.

* *Engineering News*, Vol. 60, p. 378, October 8, 1908.
† *Engineering News-Record*, Vol. 94, p. 557.

In a paper delivered before the Society in 1908* he gave his reasons for disbelieving in the value of reservoirs for flood control. He refers to a system of "automatic reservoirs" proposed by the French Government on the Rhone about 1856. Evidently, he received his information from the previously mentioned paper by Gros, a translation of which had been published.† These reservoir dams were to have open outlets, not capable of being closed, and were intended to restrain only a part of the flow. General Chittenden apparently dismissed this type of reservoir on the authority of this article.

When exactly this same type of flood control was proposed for the Miami Valley in Ohio, General Chittenden was asked to review the plans, as he was the ranking authority in opposition to reservoir flood control. After long and intensive study, he approved the Miami Conservancy program unqualifiedly, and became one of its strongest public advocates. Either the proposed French dams were not properly designed, or their limitations did not apply to certain American conditions.

Seventy-five years ago European engineers did not think on the modern scale. Methods of earth dam construction that now are fully tested were then unknown, and some sites then rejected would now be looked upon as exceptionally desirable. Projects costing $10 000 000 or $20 000 000 are referred to as too vast for accomplishment by the French Government. It was the day of small undertakings.

Later, in the paper by General Chittenden, occur these statements:

"The best reservoir site is a natural lake. * * * A natural lake, wholly uncontrolled at its outlet, may have a more effective control of the outflow than an artificial reservoir of equal superficial area when full, although of far greater capacity between high and low water. * * * if the artificial reservoir has reached the limit of its allowable filling, the outflow must be made equal to the inflow. If this limit is reached before or at the time of maximum run-off, then a quantity equal to this run-off must be let out of the reservoir. This contingency can never happen in a natural lake."

At the time General (then Colonel) Chittenden read this paper, very little study had been made of flood-control reservoirs. It is now known that the limitations of artificial regulation, which he mentions as inherent, simply represent imperfect design. General Chittenden himself fully recognized this in his studies of the Miami River problem. No natural lake is ever so good as a medium of flood control as the same body of water would be with a properly designed outlet.

Further on, in the same paper, he states:

"Every reservoir built for the purpose of flood protection alone would mean the dedication of so much land to a condition of permanent overflow in order that three or four times as much might be redeemed from occasional overflow. * * * The cost, coupled with the loss of so much land to industrial uses, would be far greater than that of levees or other methods of flood protection. * * * The construction of reservoirs for flood protection is not, therefore, to be expected, except where the reservoirs are to serve some other purpose as well."

* *Transactions*, Am. Soc. C. E., Vol. LXII (1909), p. 287.
† *Engineering News*, Vol. 25, p. 258, March 14, 1891.

Here, again, the design of flood-control dams which General Chittenden had before him for criticism was inadequate and inefficient, and has been superseded by more suitable design.

Some of the faults and limitations pointed out by General Chittenden in the design and operation of the Mississippi reservoirs in Minnesota are not inherent, as he implies, but are due to improper design or operation, or to conflict of interest resulting from an effort to make them serve more than one purpose. In his discussion of proposed reservoirs on the Sacramento and Kaw Rivers, and in his quotations from other sources, incorrect design is assumed, which largely vitiates his argument.

At the time the Miami Conservancy flood-control dams were being planned, only two flood-control dams could be found in the United States. One of these controlled a local brook, and the design of the other was so primitive and inefficient that it must largely fail of its purpose. Such was the inadequate data on which General Chittenden and his contemporaries had to work.

That General Chittenden had been unconsciously indoctrinated by the standard creed of the Mississippi River Commission on flood control, and that he only gradually freed himself, is indicated by the following comment by him when he was criticizing the traditional position of the Commission on the matter of outlets: "The writer feels the more freedom in criticizing the Committee's treatment of this matter because he is at the same time criticizing his own previous utterances on the same subject."*

After his long study of the Miami Conservancy plans General Chittenden approved the methods he formerly had condemned, and his last professional effort was a contribution to the *Transactions* of the Society pleading for an open mind on the subject.

At the beginning of the Miami Conservancy studies the writer had the same convictions as those of the Mississippi River Commission. He was certain, on the authority of the Mississippi River Commission and the Army Engineers, that reservoirs were not feasible for flood control.

On the Miami Conservancy project the possibility of reservoirs was examined simply to carry out the sound engineering policy of examining into every possibility, no matter how remote. The writer had very vigorously expressed his disbelief in reservoir flood control to the Flood Prevention Committee at Dayton. When the evidence forced him to change his views, no one was more surprised than he. At the beginning of that project not more than two or three men in a large engineering force believed in the possibility of flood control by dams. At the end every one was convinced.

Even then the writer's faith in the doctrines of the Mississippi River Commission was only slightly shaken; reservoir control could be good only on small streams. Only recently he wrote an article on the Mississippi,† in which he criticized the Mississippi River Commission, but assumed it to be substantially right in the matter of reservoirs.

In the preparation of the present paper, however, he has hunted assiduously for those deep researches and critical studies so often referred to by the Mis-

* *Transactions*, Am. Soc. C. E., Vol. LXXXI (1917), p. 1269.

† *Atlantic Monthly*, November, 1927.

sissippi River Commission and by the Army Engineers. To his astonishment, he finds they never have been made. Repeatedly there is positive assertion and the attitude of great authority and always a complete lack of any adequate engineering analysis.

Following back to original sources one finds the obsolete French article before referred to, and the report of the Engineering Board of 1874 which led to the establishment of the Mississippi River Commission. In this report, so often referred to as authoritative, may be found expressed a fundamental misconception of the proper operation of flood-control dams, followed by the following conclusive remarks:*

"The question of absolute practicability [of flood control reservoirs] could only be decided by a series of extensive and elaborate surveys, for which neither funds nor time were available, nor in the opinion of this Commission are they needed. Here, as elsewhere in the valleys, this plan, as an efficient means of restraining the floods of the Mississippi, is chimerical."

(This was the Board which estimated the total cost of complete protection by levees at $46 000 000.)† A discussion by the late W. Milnor Roberts,‡ M. Am. Soc. C. E., to prove that reservoirs are not feasible is repeatedly referred to, and is mentioned as disposing once for all of the reservoir theory. Yet, on examining this article one finds no discussion and no conception of modern flood-control reservoirs. His discussion is obsolete. Coming down from that early date to recent years one of the last affirmations of the attitude of the Mississippi River Commission may be found in the report of the Special Committee on Floods and Flood Prevention of the Society. The Chairman of this Committee was President of the Mississippi River Commission, and the report presents the orthodox Mississippi River Commission attitude, with slight concessions and some caution. In deciding against reservoirs and in favor of levees this Committee states that for rivers like the Mississippi and the Colorado, levees "afford the only sure means of flood control."§ These are the only two streams particularly mentioned, and yet, to-day, only ten years later, engineers familiar with the situation generally recognize that to a considerable degree reservoirs do offer a sure means of flood control on the Colorado.

William Kelly, Colonel, Corps of Engineers, U. S. A., M. Am. Soc. C. E., in 1924, definitely recommended reservoir flood control for the Colorado River. He states that‖: "Storage for flood control is justified to a quantity that will reduce the maximum discharge for any year of record to 75 000 sec-ft."

The writer has not presented simply a few weak points from the arguments of the Mississippi River Commission. He has presented practically all the arguments it has offered. So far as work done before 1927 is concerned, the adequate scientific study of reservoirs, so often referred to by members of the Commission, in fact is entirely a myth.

In view of the great variety of topographic and other conditions over the drainage area, it seems probable that an exhaustive study may show this

* Report of Special Board of Engineers of 1874–75, Pt. I, p. 541.
† *Loc. cit.*, Pt. I, p. 563.
‡ "Practical Views on the Improvement of the Ohio River," *Journal*, Franklin Soc., 1857.
§ *Transactions*, Am. Soc. C. E., Vol. LXXXI (1917), p. 1228.
‖ *Loc. cit.*, Vol. LXXXVIII (1925), p. 323.

method of flood control to have some proper part, if only a very limited one, in a well-considered general plan. Dependable conclusions must come from thorough study of particular cases.

In discussing suggestions that have been made by the advocates of flood control on the Mississippi by means of reservoirs, the writer finds his attitude is not that of unqualified approval. There have been many suggestions for such control, but not many of them have been worked out in detail.

One of the most carefully planned projects is that of a proposed system of reservoirs above Pittsburgh, Pa. The writer has not kept informed of that situation since the publication of the voluminous report of the Pittsburgh Flood Commission about 1910. An examination of that report, together with numerous conferences with some of those most directly responsible for its preparation, led him to the opinion that in this case the difficulties in the way of combining storage for power, flood control, and navigation in the same space would be difficult to overcome, because of inability to foretell rainfall and run-off conditions, and from the impossibility of operating gates to harmonize the demands of flood control and power development.

The difficulty of knowing when to store water for power and when to reserve storage capacity for floods is illustrated in the case of the Miami River in Ohio. In March, 1913, there was an average rainfall of 10.4 in. over the Miami water-shed above Dayton; in March, 1915, only 1.4 in. Floods come too suddenly to provide storage capacity for them by emptying reservoirs after a heavy rain has fallen, and under such variable climatic conditions it would be impossible to tell when to keep the reservoirs full for power development, and when to keep them empty for flood control. There are many cases where the two purposes can be combined, such as the proposed Boulder Canyon Dam, but popular optimism in the matter is not well grounded.

One of the chief resources of the Mississippi River Commission in its battle against reservoirs has been this weakness with which the case in favor of reservoirs has been advocated, especially by other departments of the National Government. The case in favor of reservoir flood control sometimes has been presented with an irresponsible optimism, with an unconcern for the absence of dependable information, and with a disregard of difficulties, that is disconcerting. If one is planning a military campaign it is good strategy to exploit the enemy's weakness to the utmost. Where the aim is not to overcome an opponent, but to find the truth, a different course is indicated. One of the most valuable proposals the writer ever received was from an engineer who was so visionary and impractical that he could by no means be trusted with its execution. The Mississippi River Commission might profit in the same manner from the suggestions of its adversaries.

The matter of reservoir control is a technical one and cannot be mastered except by the same kind of skillful, thorough-going, and definitely directed study that is necessary for the mastery of any difficult technical problem. No happy inspiration, no casual sallies into the field, like those of enthusiastic advocates of reservoir flood control, and no blind loyalty to a traditional cause, like that of the Mississippi River Commission, can bring dependable results. The Mississippi River Commission seems to have trusted to the advocates of

reservoir flood control to make a sufficient presentation of the case for reservoirs. The Army and the Commission have practically controlled the purse strings for the Mississippi. An adequate analysis of the problem would require years of time and very considerable funds for the accumulation and analysis of data, and the Army Engineers or the Mississippi River Commission have been the only agencies with power to secure the necessary funds.

The Commission is to be criticized, not for failure to understand the issue quickly, but for ignoring it, and for letting the years of its stewardship pass without a definitive study of the entire Mississippi problem.

In case there should be opportunities for Mississippi flood control by means of reservoirs, where might the sites be found? In the writer's opinion they most probably would occur just above the mouths of the main tributaries, such as the Yazoo, the Red, and the Arkansas Rivers, where present back-water conditions probably should be continued, and on these main tributaries and their branches, probably not more than 200 or 300 miles from the Mississippi. The greatest benefit would be expected from a few large projects, rather than from many small ones.

The writer believes that there may be opportunity for very considerable storage, amounting to many millions of acre-feet, on the Lower Ohio itself, and near the outlets of its main tributaries, as well as farther toward the head-waters. If properly designed with low-water outlets, high dams could be built without undue interference with navigation.

The St. Francis and Yazoo Basins, which are among the very best of the Mississippi bottom-lands, and perhaps other parts of the overflow area along the river, must eventually receive protection from local flood water by dams in the hills. No one of the rivers entering these basins from the hills has a capacity equal to 5% of the extreme flood flow. The dams which are necessary for flood control on these streams may have a limited but significant effect on the main river.

Retarding basins of large capacity probably are feasible on the Red River, and on other large tributaries. No human being knows what the possibilities are, because the necessary investigations never have been made.

If the facts are faced, an heroic job appears ahead, one that will involve hundreds of millions of dollars of expense. Whatever type of protection is secured must be on an unprecedented scale. Under these conditions the whole aspect of the problem is changed. From time to time the problem of flood control on the Mississippi should be examined *de novo*, as if nothing were known about it, to discover how the new scale of thinking will affect the plans. The writer believes this to be desirable, regardless of the skill and standing of the men who have made previous studies, because the state of professional knowledge and the issues to be met change so greatly that old decisions cease to be authoritative. Dams on the Ohio have been dismissed as involving too many complications. The difficulties that formerly seemed insurmountable may now be only necessary incidents of the best plan.

What would the completion of Mississippi flood control by dams and retarding basins be worth as compared with completion by further raising the

levees? Only an exhaustive study can give an adequate answer, but a few items can be suggested. Assume that the present levee system is retained, and is supplemented by dams which will hold the river within its natural banks except during years of extreme run-off, at which time the levees will come into play.

First, there are reported to be about 3 000 000 acres along the Mississippi River that are not protected by levees. The writer has not personally checked this estimate. This is recognized as the most desirable agricultural land along the river, as for the most part it is sandy loam, and not the sticky gumbo of the lower land in the basins. To reclaim an equal amount of swamp land in the Yazoo, Arkansas, or Red River Basins would cost not less than $20 per acre. This land along the river-front is high, and needs little drainage. Assuming that the land between the river and the levees should be submerged in great floods once in 10 or 20 years, such flooding would tend to renew its fertility, and it would not lose its value. With the degree of protection just mentioned, this land along the river-front would be worth probably $50 per acre more than it is now, an increase in value of $150 000 000.

High water causes most of the caving of river banks. The reduction of floods would greatly decrease this caving, and might save levee replacements and revetment work ultimately running into hundreds of millions of dollars. A credit of $50 000 000 for this item is very conservative.

The completion of interior drainage and local flood control of the St. Francis, Yazoo, and other basins, except by means of dams and retarding basins on their own water-sheds, will cost more than $250 000 000, and possibly $500 000 000. Since the Morgan Engineering Company has made plans or superintended the reclamation of more than 1 000 000 acres of this land, the writer feels that he can make an estimate that is rough, but not without value. Flood control by dams and retarding basins on the streams entering these basins, in addition to value in reclaiming the low lands of the basins themselves, would help control the Mississippi, and might properly be credited with $50 000 000.

As to the cost of complete Mississippi flood control by levees, only a guess can be made. The estimates of the Mississippi River Commission have been totally inadequate, and there are no others. Rumors are rife that the new report of the Mississippi River Commission will indicate a cost of completion of $500 000 000. Perhaps, for the first time in the history of the Commission, the estimate will be adequate. Assume that completion of control by dams and retarding basins would save further levee construction, and so may be credited with that amount, in this comparison of costs and values.

Control by reservoirs would give protection to cities on the Ohio River and perhaps on other streams. In some cases, as at Cincinnati, Ohio, control by any other means than reservoirs seems remote or impossible. This item of local protection can safely be credited with $50 000 000.

Some advocates of reservoirs have gone far wrong in assuming that there is little difficulty in combining diverse uses, such as power development, flood control, and navigation. The writer has been insistent in drawing

attention to the limitations of such combinations of use, yet those difficulties are not absolute, but are a matter of degree. Some degree of combined use is possible and desirable. Only a wild guess can be made as to the value of such a system of dams for other purposes, but an item of $50 000 000 would seem to be well within safe limits.

Finally, there would be many incidental benefits, some of which would not be immediately apparent. The higher the water rises behind levees on the Lower Mississippi the farther will it back up the tributaries, flooding the lands and interfering with drainage. Reservoir control would very greatly reduce this damage. Probably three-fourths of the silt and sand carried by the river is caved from the banks, carried a short distance, and deposited again. A more uniform flow, by reducing caving, would relieve the river of this local burden of silt, and make its silt-carrying capacity available for the permanent deepening of its bed. Much land along tributary streams and below the upper dams would be saved from frequent flood damages. The construction of dams might reduce the need for outlets or spillways. Omitting such least measurable values of reservoir control, there is a value of more than $750 000 000 to offset the cost of further levee construction. It is probable that the value would reach $1 000 000 000.

In the practice of engineering there are two fundamental processes, both of them invaluable, which should supplement, and not compete with, each other. One method is that of perfecting and refining existing methods; of letting future policies grow gradually out of past experience. The other method is that of fundamental scientific analysis and engineering design.

The breeder of race horses followed the former method and with a remarkable degree of skill, but it required fundamental engineering design and a new synthesis of elements to create the automobile. The teacher of elocution may help the orator to reach a larger audience, and may use scientific principles in the details of his work, but it required a process of far more fundamental analysis and synthesis to produce the telephone and the radio.

The employees of the Mississippi River Commission, and the Army Engineers working under its direction, sometimes have used excellent scientific methods in their work, but in its larger aspects the whole policy has been that of the practical rule-of-thumb man, and has almost entirely lacked thoroughgoing scientific analysis of the larger engineering problems. The writer believes the major policies which have governed flood control on the Mississippi River justify the following remark of John R. Freeman, Past-President, Am. Soc. C. E.,

"That the steamboat pilot on the river and the scientist in his laboratory each has certain advantage in his point of view, and that much of the river training work in America has been of the quality that might be expected to be produced by a committee of steamboat pilots, without special training in exact science."

If the problem must of necessity be assigned to either type alone, it should be to the man of practical experience, but in the present instance the country sorely needs a combination of both types of service. A member of the Mississippi River Commission recently complained of the criticism that "no

adequate scientific study has been made of this important problem."* A study of the basis of the conviction of the Mississippi River Commission against reservoirs has completely confirmed the writer in this opinion.

The improvement of the Mississippi River is under the control of the Mississippi River Commission, which includes both Army and civilian engineers. The Army Engineers assigned to district offices carry out the policies of the Commission, and are not expected to originate policies. As the experience and the duties of Army Engineers are largely concerned with administration, rather than with technical engineering studies, and as these assignments to posts on the Mississippi River are usually for short periods, they do not have time or opportunity to become qualified as experts on Mississippi River control. Under the circumstances they generally reflect the attitude of the Commission. The civilian assistants in the district offices have worked under somewhat similar limitations, except as to tenure of assignments.

The orthodox attitude of the Mississippi River Commission is the traditional attitude of the engineers along the river. In general, the influence of the Army Engineers has been to raise the standards of the Commission. It is generally believed that such spirit of research and inquiry as has existed in the Commission has largely been due to the Army Engineers. It is generally understood, also, that they have prevented very serious blunders by the Commission, such as the closing of Old River.

It is impossible to separate the history of the Mississippi from the work of the Army Engineers but it should be realized that, in their short assignments on the river and in inheriting the old standards of engineering, they have largely been the victims of circumstance. The present administration of the river is not conducive to the type of engineering analysis that is imperative.

For nearly fifty years the Mississippi River Commission has held to an arbitrary dogma of "levees only" as the final solution of this great problem. Resting on the infallibility of that doctrine, it has discouraged a thorough scientific analysis of the problem as a whole. To-day, with the estimates of half a century discredited by events, and with the public ready to furnish the resources for protection, it finds the precious years largely wasted, so far as any comprehensive research on the problem as a whole is concerned.

When a good soldier finds himself in a hard position he determines to "fight it out on that line if it takes all summer." He knows that "many a battle has been won by a bad general, but that no battle ever was won by a debating society." The hard fighting qualities of the Army officers are praiseworthy. But when a scientist finds that he has taken a questionable position his method is to examine that position to discover whether it is justifiable. There are situations in which the attitude of the scientist is no less necessary to the nation than that of the soldier.

If there should be one chance in ten that reservoirs can play an economical part in the control of the Mississippi, then it is unscientific and unprofessional to adopt an engineering plan for the control of the great river without

* *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 1480.

having thoroughly examined into that possibility. The writer believes that there is in fact a much greater chance that control by dams and retarding basins has an economical, effective, significant, and necessary place in such a policy, even if it be a small place. No more definite opinion is justified in view of the limited data available.

During the preparation of this paper he has become convinced that the examination which is necessary to determine those possibilities never has been made. He strongly believes that for the National Congress to be committed to a permanent policy with reference to flood control on the Mississippi, without such inquiry, will be a mistake of historic magnitude.

The writer was much interested in Colonel Kelly's paper.* General Jadwin, Chief of Engineers, U. S. Army, and his associates, are attacking the problem of the Mississippi with an intelligence and a comprehensive view that are promising, and command the admiration of the profession. Had that outlook characterized the study of the river for thirty years past, the present dilemma as to plans would not exist.

Doubtless great pressure will be brought upon Congress to make appropriations for reservoirs on the distant head-waters of the Mississippi in the guise of helping Mississippi flood control. The writer thoroughly agrees with Colonel Kelly that no reservoirs on the distant head-waters of tributaries to the Mississippi can have any appreciable effect on Mississippi floods. He doubts whether there is a single irrigation project in the United States on which the building of reservoirs would have any markedly favorable effect on the lower river. The Army Engineers should have the full support of the Engineering Profession in their efforts to prevent the misuse of funds through appropriations for unsound reservoir projects. This statement is a necessary introduction to any criticism of the recent study of reservoirs described by Colonel Kelly.

A criticism of that study, however, seems necessary. If the analysis of the entire Mississippi problem, so intelligently outlined and undertaken in June, 1927, by the Chief of Engineers and his associates, could have been spread over a number of years, the country would be fortunate. To endeavor to crowd into six months the work of ten years is not productive of dependable results. The writer believes this is true of the recent study of reservoirs described by Colonel Kelly, for the following reasons.

Until recent years there has been little understanding in America of the principles of design and operation of flood-control dams. In the search by Army Engineers some years ago for flood-control reservoir sites in Ohio, not one of the sites used on the Miami Conservancy project was discovered. Every one of the most significant possibilities in the State was overlooked. In the development of the Miami Conservancy plans the writer found that widely experienced and competent hydraulic and power development engineers, who had not thought in terms of flood control, often had very erroneous conceptions of the problem. No hasty examination would have developed the reservoir possibilities of that project. The actual necessities and possi-

* See p. 724.

bilities could be discovered only by a process of thorough and fundamental analysis and design.

The control of the Mississippi by reservoirs or by any other means will require engineering imagination and creative design on an unprecedented scale, and only persons especially prepared, and having a clear understanding of the problem as a whole, would have a concept of the nature and functions of such reservoirs, or would recognize the existence of possible sites. The writer does not believe it is possible to send instructions to a miscellaneous group of division engineers, some of them burdened with flood-relief work, whose past duties have been largely administrative, few of whom have had any experience with flood-control reservoirs, or have given them any thought, and to get hastily prepared reports of very much value.

It is understood that instructions for the examination of reservoir sites were sent to the various division engineers about July 1, that preliminary reports were requested in about two weeks, and final reports about six weeks later, or about September 1. More than 400 000 sq. miles of territory had to be reported on in that time, only a minor part of which was covered with topographic maps. Under these conditions there was small chance that the significant sites would be recognized and properly appraised. To assume that errors would cancel each other is somewhat like giving a mathematical problem to several men who do not understand it, and taking the average of their answers as correct.

The writer believes that the rule adopted, namely, to consider that reservoir gates would be closed on February 1 for three months, is an arbitrary and improper assumption, made necessary by the great haste in which the work had to be done, and that this rule alone would largely prevent the recognition of favorable sites.

It is believed that this study will be worse than useless if it leaves the impression that an adequate investigation has been made, and if the possibility of a material degree of control by reservoirs is thereby dismissed without adequate examination. After reviewing Colonel Kelly's paper, the writer is of the opinion that an adequate study of this problem still remains to be made.

In his paper* Colonel Townsend, former President of the Mississippi River Commission, stated that in the further raising of the levees the cost would increase approximately as the square or the cube of the height. The last foot of raise is far the most expensive. Colonel Kelly, on the other hand, correctly states that, if reservoir control is sought, the top foot of water is most easily held back, because the most economical reservoir sites are first chosen. Thus, the work which can be done most economically by reservoirs is that which is most expensive if accomplished by levees. The writer believes it is very probable that at some point, to be determined only by very thorough-going analysis, the curves of economy will cross, and the balance will be in favor of reservoirs.

The confusion of counsel and the conflict of opinion running all through the present discussion of this subject is not due to absence of engineering

* See p. 666.

intelligence, but to a lack of the information necessary for sound judgment. If the Mississippi River Commission for the past thirty years had made the necessary studies, these controversies would already have been settled by the facts.

On the Miami Conservancy project there were at first the same conflicts of opinion, but the technical studies of the district settled nearly all mooted questions, and the technical design adopted has been generally approved by American engineers. Similar unity concerning the Mississippi problem will follow a proper study of the facts. No other policy than long-continued, thorough-going, comprehensive analysis will bring that unity. The country should not be committed to a far-reaching flood-control policy without such study. At some time, the fact of lack of information must be acknowledged, and this policy of thorough-going analysis adopted. It will then take years of such study to arrive at a dependable decision.

In conclusion, the writer would emphasize the fact that he is not an advocate of any specific degree of reservoir control as a proven method of flood control for the Mississippi. He does not know to what extent such control is feasible; but he does advocate a deliberate and conclusive study of the subject, which, he believes, never has been made.

LEVEES AS A MEANS OF FLOOD CONTROL FOR THE MISSISSIPPI RIVER

By J. F. Coleman,* M. Am. Soc. C. E.

The control of the floods of the Mississippi River is truly a great engineering problem. It involves uncertainties and difficulties to a greater extent than did the Panama Canal, or any other great work of which the writer has knowledge.

Control of these floods on a basis of any method which has been suggested would involve the expenditure of great sums of money which have not hitherto been available. For this reason the task of those who have been responsible has been to expend such funds as could be obtained in such manner as would afford the most protection. In the main, they have performed this task creditably. Political expediency and sectional jealousy have played their parts in impeding the appropriation of sufficient funds with which to accomplish the task.

By reason of the vast territory and large population whose interests are so greatly involved, the problem has received the attention of many men, both technical and non-technical, some at least of whom have not hesitated to proclaim full and final solution with little or none of the fundamental data on which a true solution must be based.

There can be no better forum than the Society before which to discuss this great engineering problem. The subject has been before it on past occasions and no doubt will be again until the true solution is found. The problem is far more complex than appears upon superficial examination and it is not to be expected that the ultimate solution will be attained without research, study, and analysis, such as would be applied to other engineering problems. It is unfortunately true that an enormous amount of misinformation is extant, which makes it necessary for the student to check carefully a great many of the data which he will assemble before reposing too much confidence therein.

As in the study of other problems in river hydraulics, one is soon brought to a realization of the vagaries and eccentricities of water flowing in open channels of the character of river channels, such, for example, that a given cross-sectional area and gauge reading does not always produce the same velocity and discharge; or that a given gauge reading at one point on the river does not always produce another given gauge reading at another point lower down; or that the flood-plane of the river along any given part of its length is a broken line, and that there is frequently an appreciable variation between the line showing the flood-plane at one time and that of another time when the river stage is approximately the same, etc.

The various formulas for the flow of water in open channels, which have served useful purposes in smaller streams and in canals, often serve to mislead when applied to so large a stream as the Mississippi River.

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Necessarily some of the factors to be dealt with may not be definitely known, and for practical purposes must be assumed. The most important of these is the volume of water for which provision must be made. The volume which has flowed down the main river channel during each of what may be called the high-water years for the last thirty or forty years is known with close approximation; but there has never been a time when all the tributaries of the main river have been in maximum freshet simultaneously, or rather at such time as to deliver their respective flows into the Mississippi River in synchronism with the wave crest of the flood in that river.

It is probable that there never will be such a time. It also seems probable that the flood of 1927 will never be exceeded. On the other hand, it is possible for the river to be called on to handle a greater flood than that of 1927. This flood problem is a very vital one to the residents and property owners of the Mississippi River Valley and to those of the valleys of many of its tributaries. Perhaps there has never before been a time when it has received the attention of as many engineers as are now concentrated upon it; and it is probably true that a greater amount of information is now, or soon will be, available than has heretofore existed.

All the methods proposed or suggested for controlling the floods of the Mississippi River include levees; and all of them also include revetment for the control of caving banks. The writer is among those who believe that levees and bank revetment, without the aid of reforestation, contour plowing, reservoirs, outlets, or spillways, will most surely provide the necessary control for the floods of the Mississippi River. This river, it will be remembered, is an alluvial stream flowing in a bed of its own creation. Its burden of silt is heavy and its need for energy with which to transport its silt-laden waters is great. The channel, as it now exists, is admittedly capable of carrying a volume of about 1 000 000 sec-ft. at a bank-full stage from Cairo, Ill., to the Gulf of Mexico. The maximum discharges recorded prior to 1927 were, as follows:

Place.	Year.	Second-feet.
Columbus, Ky.	1912	2 015 213
Helena, Ark.	1912	2 040 660
Arkansas City, Ark.	1912	2 006 601
Vicksburg, Miss.	1922	1 825 555
Red River Landing, La.	1912	1 499 402
New Orleans, La.	1922	1 358 311

The flood of 1927 is said to have been the greatest on record. There is, however, a divergence of opinion as to its volume. Estimates range from 2 000 000 sec-ft. to as much as 3 250 000 sec-ft. Whatever may have been the actual volume it would surely have been more than 2 000 000 sec-ft. if entirely confined within levees, which would have been more than double the capacity of the bank-full river. It is apparent, therefore, that the channel without levees is not large enough to carry the burden in time of freshets and must be enlarged.

A part of this necessary enlargement is taken care of by the leveed section above the bank-full stage. More may be provided by enlargement of the section below that stage.

The "Levees Only", or "Confinement", plan is based on the theory that, in an alluvial bed or channel, the energy and power required for channel enlargement may best be obtained by confining the flowing water so that its velocity will be increased and its scouring capacity will be put to the useful purpose of enlarging the channel.

According to the same theory, the removal or diversion of any part of the waters from the main stream reduces the velocity (and, consequently, the energy) of the main stream below the point of divergence and results in deposits of silt and the reduction of cross-section of the main channel, and hence reduces its carrying capacity; or, to state the proposition in the language of the first report (1880) of the Mississippi River Commission:

"If the normal volume of water in a silt bearing stream flowing in a bed of its own formation be permanently increased, there will result increase of velocity and consequently of the erosive and silt bearing power; an increase of depth, if the banks are held, and an ultimate lowering of the surface slope; and conversely if the normal flow be decreased in volume, there will ensue a decrease in velocity, silt transporting power, and mean sectional area, and an ultimate raising of the surface slope."

Let us examine how this theory fits the records of such streams of this character as are available.

In 1913, the late J. A. Ockerson, Past-President, Am. Soc. C. E., compiled experiences and conclusions of eminent authorities as to the utility of outlets,* from which the following excerpts are quoted:

"(1) Result of the diversion of the Adige into the Castagnaro, Italy.—Elevation of flood level by divided flow and conversely a lowering of flood level when waters were restored to one channel.

"M. R. C. 1890, p. 3106:—'About A. D. 1438 the Adige broke its levees and poured its waters south into the Castagnaro and Canal Bianco, which then formed a drainage stream parallel to the Po. In 1545 the break had so increased, that two-thirds of the low-water flow of the Adige, and three-fourths of the high-water flow, went through it. A low dam was built across the Castagnaro to check the flow into it, and both rivers raised their beds. In 1678 a new dam was built, as the old one was then buried in the deposit. The bed still rose. In 1791 a masonry dam, 39 feet high, with many arch ways through it, to allow floods to pass, was built across the Castagnaro. The bed continued to rise and the floods on the Adige were so high that in 1838 the Castagnaro was permanently closed. In the six years following the closure the floods in the Adige fell, and the more markedly the nearer the point considered was to the Castagnaro.'

"(2) Extracts from work of Frizi, an eminent Italian Engineer, published in 1762.

"Of the Rhine he says: 'The great multiplicity of channels, though productive of very great advantages to the navigation and commerce of Holland, draws after it very fatal consequences. The waters divided into so many branches lost the rapidity and strength which are required to sustain and push forward those heterogenous substances which they transport. The con-

* "Outlets for Reducing Flood Heights", p. 14, pub. by Mississippi River Comm., 1914.

stant rising of the bottom renders the drainage of the waters from the fields more difficult, increases the expense of the necessary embankments, and always augments the damage which these extensive lowlands suffer when the dikes break and threaten the whole country in ruin.'

"To remedy that trouble it was proposed in 1754 to provide still more channels and sluice ways. Gennette, however, maintained that this would not diminish the height of floods; that all of the several channels should be abandoned save one through which the waters should be carried to the sea by the shortest practicable route, and his advice was followed with satisfactory results.

"The effect of adding to or subtracting from the volume of a river is described as follows: 'We read in the collection of observations for 1728 that, having made the experiment of placing a mark (gage) in the Panaro (river) and of letting in and afterwards withdrawing the waters of the great drain of Burana, they observed in the Panaro no sensible rise in the first instance, nor any visible decrease in the second. These three facts have been particularly attested by Eustice Manfredi, whose testimony is worth that of all others. No objections can be made to these facts, for it cannot be said that the quantity of water in the affluent bore no sensible proportion to that of the recipient stream, nor that the sections of the recipient were not effective; neither can the variability of these very sections be attributed to any other cause than an increase of velocity in the united waters, proportioned to the increased quantity of water itself.'

"'* * * In diverting from the principal channel a considerable volume of water, that which is left behind is not visibly diminished either in height or in breadth.'

"Other early observations on the Po confirm this principle.

"'It is a hydrostatic paradox, commonly taught by Italian authors, and uniformly confirmed by experience, that you do not diminish the height of the waters in great floods by lessening the quantity of water.'

"Father Castelli, in the 13th Corollary of his first book on Running Waters, disapproved of the division formerly made of the Po at Buondens and which was afterwards abandoned. (In 1838).

"Guglielmini confirmed the opinion as to the little utility of spillways for reducing flood heights in streams.

"Eustice Manfredi proved the futility and danger of spillways proposed for the right bank of the Serchid River. Experience in like manner demonstrated the futility of a cut made in the right bank of the Arno to prevent the flooding of the City of Pisa. This cut was made in 1740 but no perceptible diminution of flood height at Pisa occurred. The cut was made anew in 1761 during a great flood but the waters continued to rise so high that the people 'could not be persuaded that the cut had been made.' In fact, the greatest flood within the memory of man was recorded.

"Cornelius Meyer, a celebrated Dutch engineer, disapproved of this method of dissipating the flood, and instead proposed confining the water to a single channel with ample embankments to carry the flood waters safely to the sea.

"'The canal made by order of the Emperor Nerva to draw off the superfluous waters of the Tiber at the times of its great freshets, did not contribute in the smallest degree to prevent inundations.'

"One may also inspect the discourse of the celebrated Lorgna on the inundations of the Adige, which sufficiently proves that all the derivations made in that river have only produced a heightening of its bed, and thereby rendered its floods more dangerous.

"The following rules, as set forth by Guglielmini, govern the regimen of alluvial streams:

"'The greater the ordinary body of water in a river the less will be the slope of its bed.

"'The greater the quantity of water which a river carries, the less will be its fall; and the greater the force of the stream, the less will be the slope of the bed.

"'These two rules resolve themselves into a single rule, viz., That the slope of the bottom in rivers will diminish in the same proportion in which the body of water is increased.'"

In the Annual Report of the Mississippi River Commission for 1881, page 128, occurs a statement to the effect that levee building on Red River below Shreveport, La., began about 1860; and that the bed of the river has been depressed and "lands have not been inundated for years."

The writer has not been able to obtain actual records of the surveys made of Red River since 1890, and, hence, is unable to give exact figures. Mr. Gervais Lombard, a member of the Board of Louisiana State Engineers, who has been in charge of levee work on Red River for many years, states that below Shreveport the size of the channel has increased about 300%; that the bed of the river is generally lower in elevation; and that a flood of given volume is carried out of that river now with a lower flood-plane than was possible prior to the completion of the levee system.

The Upper Yazoo River Levee District has enjoyed absolute protection from floods for a number of years. That District has been fortunate in having adequate funds with which to build superior permanent levees and auxiliary works. The levees have a free-board of about 5 ft.; are heavier than the standard; have a hard surface roadway on the banquette; and are systematically and effectively maintained by regularly employed section gangs. The writer is not able to furnish information in specific terms as to the size of the existing channel by comparison with the channel at the time of the completion of the levee system. It is understood that the channel has enlarged itself.

On the Mississippi River many opportunities have been lost for gathering and recording valuable data in precise terms. This has been due, no doubt, to a lack of sufficient funds, and perhaps, in part, to a failure to appreciate the value of such data.

Many outlets and spillways have been closed; many crevasses have occurred, some of which remained open for several years, but all of which were ultimately closed. Neither in the case of the closed outlets and spillways, nor in that of the crevasses, has there been available accurate surveys of the river before and after. It is known that just below each outlet and spillway the river contained shoals and sand-bars which were obliterated after the closure. It is also known that below each crevasse the river has been reduced in section and that, when the crevasse remained open over a period of years, this reduction has been progressive; also that upon the closure of the crevasse the reenlargement of the cross-section immediately began.

For lack of actual surveys, it is not possible to furnish precise figures of the reduction or of the enlargement resulting from any crevasse and its

closure. The only outlet which still remains open (other than the yet unclosed crevasses of 1927) is Old River (just above Red River Landing), through which large volumes of water leave the Mississippi River in high-water periods and pass through the Atchafalaya River to the Gulf. Surveys have been made of the Mississippi River in the vicinity of this outlet from time to time over a period of years.

Cross-sectional areas based on "low water" at Torras Landing and at Red River Landing, respectively, about ½ mile and 1 mile below the outlet, are given approximately in Table 28.

TABLE 28.—Cross-Sectional Areas, in Square Feet.

Place.	Year.				
	1884.	1908.	1918.	1922.	1924.
Torras Landing.....	61 300	21 150	21 300	19 900	23 600
Red River Landing.....	73 800	30 500	29 850	36 200	30 500

For some years past it has been necessary to dredge the channel in low-water seasons in the interests of navigation, and it is not known to the writer whether or not this dredging may have had the effect of maintaining a fairly constant sectional area from 1908 to 1924. Later surveys are not yet (1927) available. It is noteworthy, however, that the cross-sectional areas from 1908 to 1924 are only about 35 to 40% of the cross-sectional areas of 1884.

It seems safe to assert that a crevasse, a spillway, or an outlet will surely result in a progressive reduction in the main channel a short distance below the said crevasse, spillway, or outlet, and that this progressive reduction of channel will continue as long as the diversion of water from the main stream continues. There is also much evidence to support the belief that the crevasses and spillways have a tendency to silt themselves up gradually and, therefore, to grow less and less efficient as the years go by, although the evidence is not in the form of actual surveys.

If technically and practically the "Confinement" theory is right, as the writer believes it to be, it will be necessary either to close the mouth of Old River and confine all the waters of the Mississippi River within the channel of that stream, or to close the Mississippi River just below the mouth of Old River and divert the waters to the Gulf of Mexico through the Atchafalaya River. The latter route is shorter by about 150 miles, but the volume to be provided for would be greater than in the former case by the addition of waters from Red River and its tributaries.

It is not the purpose of this paper to select the channel, or to enter into questions of details of levee construction or bank protection. It seems to the writer that the first step to take in a study of this great problem is to determine, with as much certainty as possible, whether it is better to concentrate the flood waters in one channel, or split these waters into two or more channels by maintaining the Old River outlet and by creating other outlets

and spillways. When the broad general principle is settled, it will be time enough to attempt the determinations of other points.

The unquestioned fact that the stream of large sectional area will attain greater velocities and consequently greater discharge capacities with a given hydraulic slope than would be possible with smaller streams of equal total sectional area is a strong point in favor of the "Confinement" theory. The increased velocities resulting from confinement of the flood waters and the increase in silt-bearing capacity and in scouring capacity, which follow such increase in velocity, hold forth promise of the enlargement of the channel by the erosive forces of the contained waters, and an enlarged channel is needed to carry the excess burden of freshet waters.

The knowledge that the diversion of waters by outlet, spillway, or crevasse will surely check velocities and cause deposits of silt, and hence will reduce the cross-section of the main channel, foreshadows ultimate reduction of carrying capacity as a natural sequence.

It cannot be questioned that a crevasse affords an immediate relief when the river is surcharged and the containing levees are water-soaked or insufficient in elevation or stability; but every crevasse also results in a depletion of the capacity of the main channel, which needs to be enlarged rather than reduced.

If the problem were merely to deal with the control of floods for a few years, perhaps spillways or outlets would serve; but the problem is to provide for all posterity, and it appears to the writer that the advantages which may be gained by spillways are not only temporary, but that they are at so enormous a cost for the future that it would be an engineering mistake to formulate designs of flood control based upon a system of spillways.

For the reasons stated, the writer is of the earnest conviction that levees, with auxiliary works for the protection of banks from caving, will afford the only certain method of controlling the floods of the Mississippi River.

IMPROVEMENT OF NAVIGATION IN RELATION TO FLOOD CONTROL

By Stuart C. Godfrey,* M. Am. Soc. C. E.

I.—Navigation and Flood Control

"And the waters prevailed, and increased greatly upon the earth; and the ark went upon the face of the waters." This passage from Genesis is the earliest record that the writer finds of the relationship of floods and navigation. In the modern parallel that occurred in April, 1927, Noah's one ark had increased to a hundred, freely and instantly made available by the Army, the Navy, the Coast Guard, the Lighthouse Bureau, and the various commercial boat lines. In place of the dove, fortunately, the seaplane and the radio were available. And the outcome proved anew, if proof were needed, that these boats cannot be spared from the rivers.

To-day, on the Lower Mississippi, navigation is overshadowed in importance by flood control. The latter, like a modern prodigal son, after a season of wildness and dissipation, comes back to be the center of popular attention. Navigation is the elder brother who once enjoyed pre-eminence on the river, but who now finds himself relegated, at least for the present, to a subordinate place at the feast. All this is natural enough; a wild animal must be tamed before he can become a useful beast of burden. And the elder brother may accept his position the more gracefully, inasmuch as his brother's salvation will bring to him also great benefits.

II.—Present Extent of Navigation

Indeed, navigation on the Mississippi River does not lack encouraging developments. The present volume of this navigation is not generally appreciated. Consider the giant steamer, *Sprague*—so useful in April, 1927, in rescue activities—at her usual task, supplying the Standard Oil Refinery, at Baton Rouge, La., with crude oil from the Smackover Fields. On March 21, 1926, the *Sprague* finished a 300-mile tow at Baton Rouge with a cargo of petroleum (224 000 bbl. of crude oil) contained in 19 steel barges. (See Fig. 4.)

This tow, the size of four city blocks, contained more than could be carried in the largest tank steamer; the equivalent of no less than 28 full train loads of 40 cars each, or one solid train 9 miles long. It would have taken about thirty of the largest old-time packet-boats to carry this one cargo. No wonder the rivers do not appear so busy in this era of barges and towboats! Yet the Secretary of War was able to state in his Annual Report for 1926 that the tonnage now handled on the Mississippi River System is twice what it ever was in the "good old days" of river steamboating.

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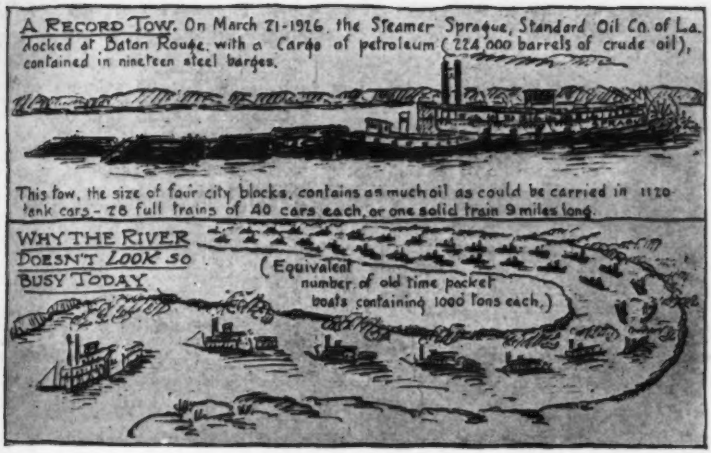


FIG. 4.—VISUAL COMPARISON OF PRESENT AND FORMER TRAFFIC HANDLING.

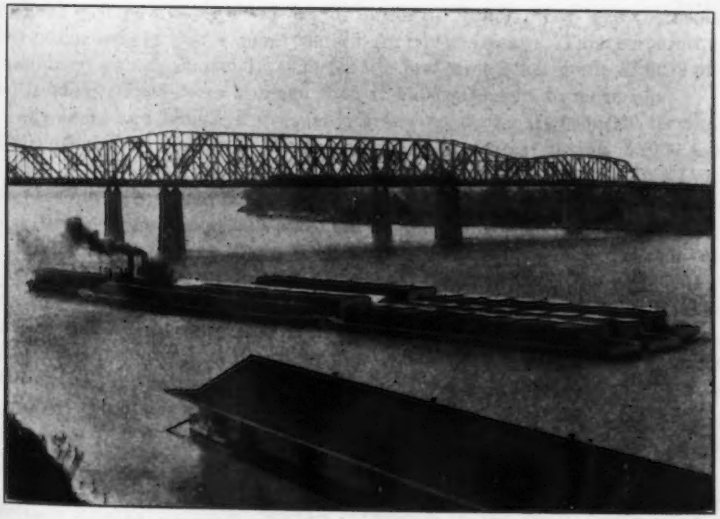


FIG. 5.—TYPICAL FEDERAL BARGE LINE TOW.

The single traffic in crude oil referred to, handled by the *Sprague* and half a dozen other towboats, amounted in 1926 to 2 500 000 tons, about the equal of the entire foreign commerce of Boston, Mass., Newport News, Va., or Galveston, Tex.

The tonnage on the Mississippi and Ohio Rivers has increased greatly since 1920, as shown by Table 29.

TABLE 29.—SOME COMMERCIAL STATISTICS.

	ANNUAL TONNAGE.		
	1920.	1923.	1926.
Cairo, Ill., to Memphis, Tenn.	1 044 945	1 048 322	1 660 188
Memphis, Tenn., to Vicksburg, Miss.	925 763	1 452 837	4 792 780
Vicksburg, Miss., to New Orleans, La.	2 874 190	5 493 297	11 074 488
Ohio River	9 869 325	9 245 647	19 700 000*

* Approximately.

The Inland Waterways Corporation, in its Mississippi-Warrior Service (operating on the Lower Mississippi and Warrior Rivers), charging a rate which is 80% of the rail rate, carried 1 341 000 tons of freight in 1926, an increase of 18% over the preceding year, with a direct saving in rates to shippers of more than $2 000 000. (See Fig. 5.) This season (1927) the Service has been extended to the Upper Mississippi, with three new towboats of lighter draft now in commission. Promising experiments in the use of powdered coal now being tried on the towboat, *Illinois*, may lead to revolutionary changes in power plant design for river vessels. This corporation, Government-owned, showed in 1926 for the first time a net profit of $219 512, and is being offered more tonnage than it has equipment to move.

The Jones and Laughlin Company is now enlarging its $350 000 terminal at Memphis, Tenn., where it handles annually, at a great saving, 50 000 tons or more of steel products shipped by river from the Upper Ohio. This Company has not only issued an attractive and illuminating pamphlet, "Our Runaway Rivers—Put Them to Work", but is practicing what it preaches.

The Carnegie Steel Company is one of several concerns that are making an increasing use of the Ohio and Mississippi for the shipment of heavy tows of steel from the Pittsburgh District by water to the Gulf. This through traffic can be expected to increase greatly with the completion of the Ohio River improvement project by 1929. The W. C. Kelly Barge Line has built and is putting into operation a fleet of modern Diesel towboats of great capacity. Even the old packet-boats, in general an outgrown type, have in some cases found a successor in such fine modern passenger boats as the *Cincinnati*, plying between Cincinnati, Ohio, and Louisville, Ky.

As for terminals, there is a growing appreciation of the vital need of adequate terminal facilities, finding expression in the rapid development at numerous sites of such structures as the river and rail terminal at Memphis, representing an investment by the City of more than $2 000 000.

The economics of transportation on these rivers, especially the Ohio, has been discussed* before the Society, and will not be here reviewed. However, the symptoms referred to, relating to navigation at present, are not those of a languishing patient. Rather do they give encouragement to President Hoover's vision of a 9 000-mile consolidated system of inland waterways serving twenty States, developed to keep pace with the vast expansion of transportation facilities that will be required in the next twenty-five years.

III.—Improvement of Navigation on the Lower Mississippi

The scope of this paper precludes more than a brief discussion of some of the practical aspects of maintaining a navigable channel on the Lower Mississippi. The Mississippi from Cairo to the Gulf is about 1 060 miles in length; the lower 290 miles, below the mouth of the Red River, has ample depth for river craft at all times.

Among the factors which make the Lower Mississippi difficult for navigation are the instability of its channel, the swiftness of its current in places, its easily eroded banks, and the constantly shifting bars. The bar-forming material is supplied from the eroded banks, which give up annually to the river nearly 1 000 000 000 cu. yd. The most favorable factor, as compared with other rivers, is the large low-water discharge, which seldom falls below 100 000 sec-ft.

The general formation of the river is not essentially different from that of other alluvial streams, with abundant water in the bends, which are separated, however, by long bars stretching obliquely across the river. At the broad "crossings" over many of these bars, the same river that at flood stages measures 60 ft. or more on the gauge, fails by its own efforts to maintain a low-water channel more than 4 or 5 ft. deep. The location of these bars shifts from season to season, and even from day to day. The one unchanging thing about the river is its changeableness.

The first comprehensive plan for improving the Mississippi River was by means of regulation. In 1880, when the Mississippi River Commission made its preliminary report, river navigation was a controlling factor, and the plan of improvement comprised the contraction of the waterway and the protection of caving banks, in order to scour out and maintain a channel through the shoals and to build up new banks and establish a fixed regimen. It was then the general belief, based upon experience with other rivers, that such results might be secured largely through the instrumentality of light flexible structures of poles and brush.

The initial works of river regulation undertaken by the Commission, and those most often referred to, were located in the Plum Point Reach, and the Lake Providence Reach, where low-water navigation was particularly difficult. On these works there was expended over a period of several years between $4 000 000 and $5 000 000, more than two-thirds of which went into contraction works.

* *Transactions*, Am. Soc. C. E., Vol. 89 (1926), p. 1105.

These works improved navigable conditions, and confirmed the soundness of the general theory of improvement. The frail structures of poles and brush, however, proved too light to withstand long the ravages of the floods. Too little attention relatively was paid at that time to bank protection, and most of the contraction works, lacking continuity and completeness, failed sooner or later through changes in the river resulting in "flanking," if not by direct attack. (See Fig. 6.)

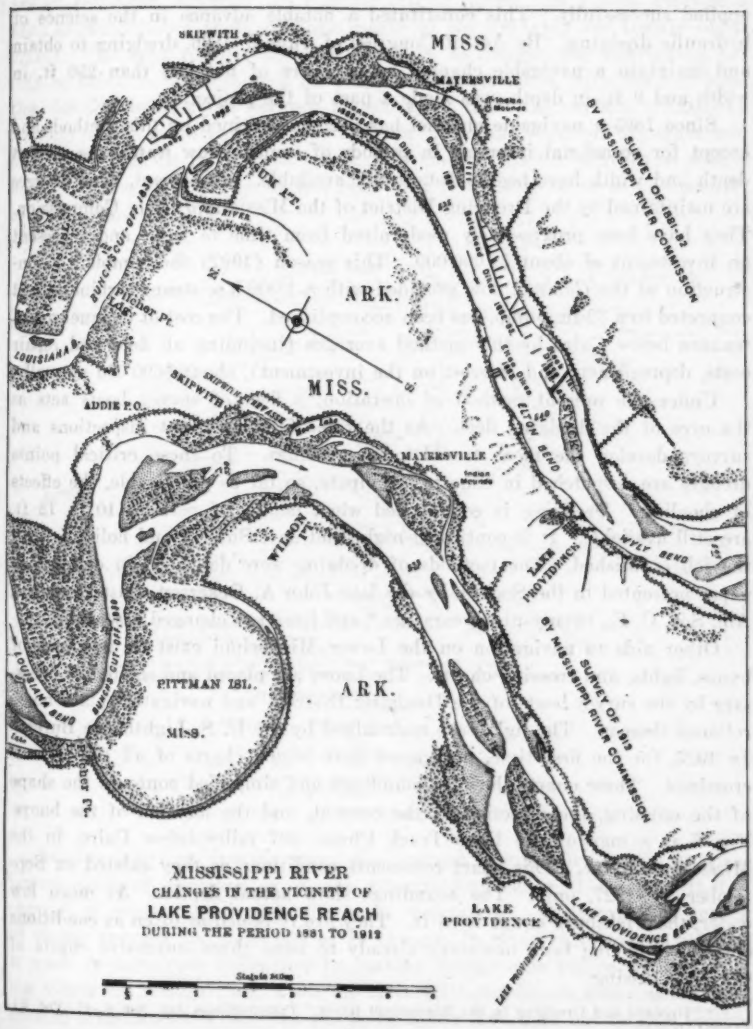


FIG. 6.

Channel Maintenance by Dredging.—When it became apparent that channel improvement by contraction works, as then conceived, could not meet the pressing needs of commerce within a reasonable time limit, the Commission began experiments with hydraulic dredges of large capacity. After extended study and experiment, such dredges adapted to this peculiar service were developed, and the plan for the temporary improvement of low-water navigation by opening and maintaining channels across the obstructing bars, was applied successfully. This constituted a notable advance in the science of hydraulic dredging. By Act of Congress of June 3, 1896, dredging to obtain and maintain a navigable channel below Cairo of not less than 250 ft. in width and 9 ft. in depth, was made a part of the project.

Since 1895 a navigable channel has been maintained by this method, and except for occasional intervals in periods of unusual low water, the project depth and width have been substantially available. At present, nine dredges are maintained by the Dredging District of the Mississippi River Commission. They have been progressively modernized from time to time, and represent an investment of about $2 000 000. This season (1927) the complete reconstruction of the *Gamma,* now provided with a 1 000-h.p. steam turbine, direct connected to a 32-in. pump, has been accomplished. The cost of channel maintenance below Cairo by this method averages (including all field and repair costs, depreciation, and interest on the investment), about $600 000 annually.

Under the present method of operation, a fleet of survey boats acts as the eyes of the dredging fleet. As the river falls, frequent inspections and surveys develop the more troublesome crossings. To these critical points dredges are despatched in time to anticipate, so far as practicable, the effects of shoaling. Dredging is commenced when depths of perhaps 10 to 12 ft. are still available. It is continued night and day, Sunday and holiday, until the job is finished. The methods of dredging were described in detail in a paper presented to the Society by the late John A. Ockerson, Past-President, Am. Soc. C. E., twenty-nine years ago,* and have not changed essentially.

Other aids to navigation on the Lower Mississippi exist in the form of buoys, lights, and crossing charts. The buoys are placed and shifted as necessary by the survey-boats of the Dredging District, and navigators place much reliance thereon. The lights are maintained by the U. S. Lighthouse Bureau. In 1927, for the first time, navigators were issued charts of all troublesome crossings. These charts show by soundings and simplified contours the shape of the crossing, the direction of the current, and the location of the buoys. Fig. 7 is a map of the Race Track Chute, 607 miles below Cairo, in the Mississippi River. This chart represents conditions as they existed on September 24, 1927, only. The soundings show actual depths. At mean low water, the Vicksburg gauge is 3½ ft. They are re-issued as often as conditions change, it having been necessary already to issue three successive charts of Corona Crossing.

* "Dredges and Dredging in the Mississippi River," *Transactions,* Am. Soc. C. E., Vol. XL (1898), p. 215.

Limitations of Dredging.—In this connection, it should be understood that the dredging done for channel maintenance is purely temporary in nature. The excavated material is not removed from the river, but deposited a few hundred feet down stream from the channel limits. If the location of the dredged cut has been skillfully chosen, the channel will maintain itself during the remainder of that particular low-water period. At the first considerable rise of the river, however, the channel fills up much like a path through a bank of wind-driven snow, and the marks of the dredging are largely obliterated. The next time it becomes necessary to dredge, the obstructing bar may be found to have assumed an entirely different shape, with the new location for the dredged cut perhaps a mile away from the old site. Thus, no stability exists as to channel location, nor as to the location of lights and buoys. So difficult is the pilot's task that one will hardly venture to take a boat down the river unless he has been over it recently so as to be "posted" as to its changes.

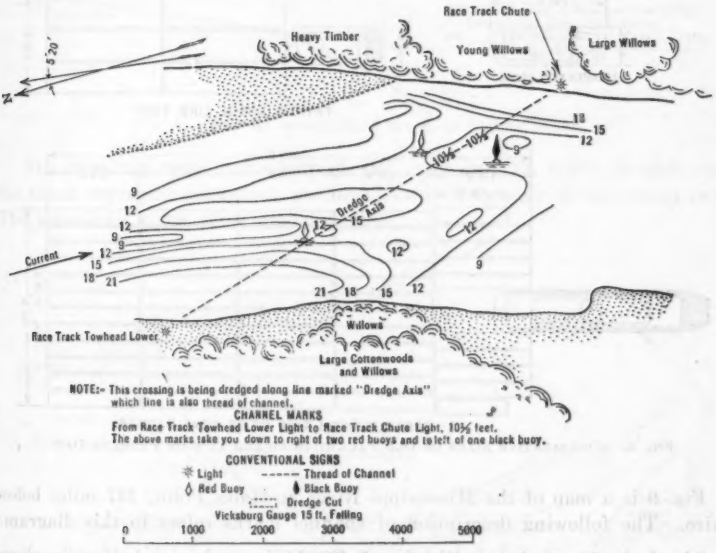


FIG. 7.—RACE TRACK CHUTE.

Another inherent disadvantage of a dredged channel lies in the physical impossibility of ensuring, with any reasonable number of dredges, that a channel of project depth will always be maintained. Ordinarily, by means of timely surveys and forecasts as to changes in river regimen, an attempt is made to anticipate difficulties by making dredge cuts before the critically low stages are reached; but when the river falls rapidly, so many shoals may develop simultaneously that the entire dredging fleet may not be able to give timely assistance at all needed points.

Another serious objection to the present project lies in the restricted width of channel. For the old-time packet-boats a channel width of 250 ft. was perhaps sufficient. The situation has entirely changed with the advent of the present type of towboat and barges. (See Fig. 8.) The normal tow for a Federal Barge Line towboat, for example, is six barges, three abreast, with outside dimensions of 135 by 750 ft. The Standard Oil tows are even larger. Only when such a channel is straight and parallel with the current can it be regarded as affording adequate width. In the occasional sharp bends (see Fig. 9), curved channels (Fig. 10), or those with the current running obliquely to the channel, so as to require "flanking" on the part of navigators, the channel becomes hazardous in the extreme. In such cases, the pilots of large tows usually resort to "double tripping," involving great loss of time.

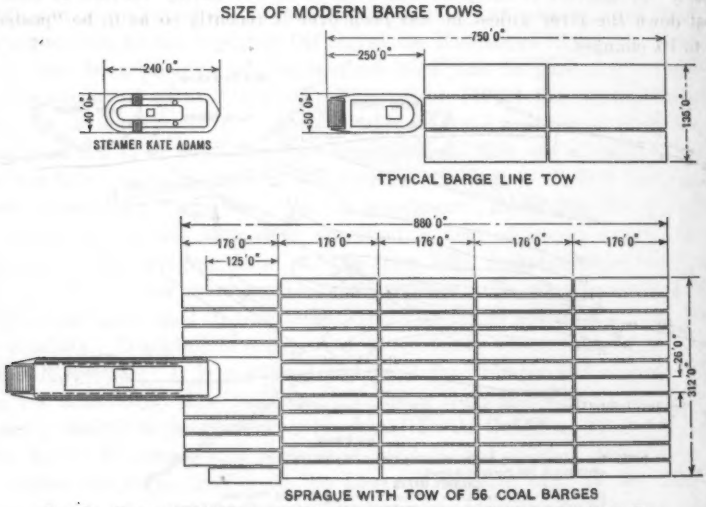


FIG. 8.—COMPARATIVE SIZES OF OLD PACKET-BOAT AND TOW OF PRESENT DAY.

Fig. 9 is a map of the Mississippi River at Hales Point, 137 miles below Cairo. The following description of channel marks refers to this diagram:

"At from 50 yd. below Wrights Point Light to target half mile above Hales Point Light. This takes you 75 yd. to right of red buoy and 100 yd. to left of first black buoy and 25 yd. to left of second black buoy, 13 ft., one cast until from Tennessee Point opposite Island 21 to tin roof barn 250 yd. below target, 15 ft. This takes you to left of third black buoy."

Fig. 10 is a map of the Mississippi at Corona, 204 miles below Cairo. The corresponding channel marks are described as follows:

"From head of white sand below Massey light to 150 yd. below Corona upper light, 13½ ft. Down this way till from 150 yd. below Corona Bar light to Corona lower light, 15 ft. The above marks take you down to right of two red and left of four black buoys, favoring the red buoys."

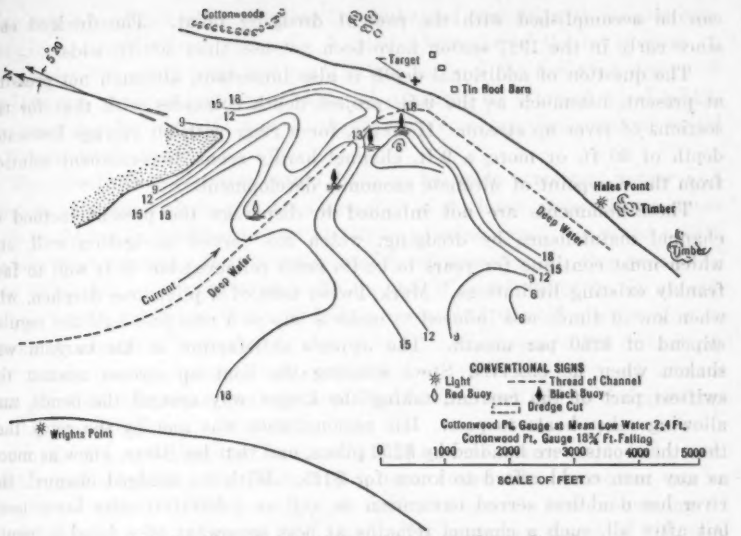


FIG. 9.—MAP OF MISSISSIPPI RIVER AT HALES POINT.

The soundings were taken July 26, 1927, and July 23, 1927. In each case, the charts represent conditions as they existed on the day of the survey only. The soundings given show actual depths.

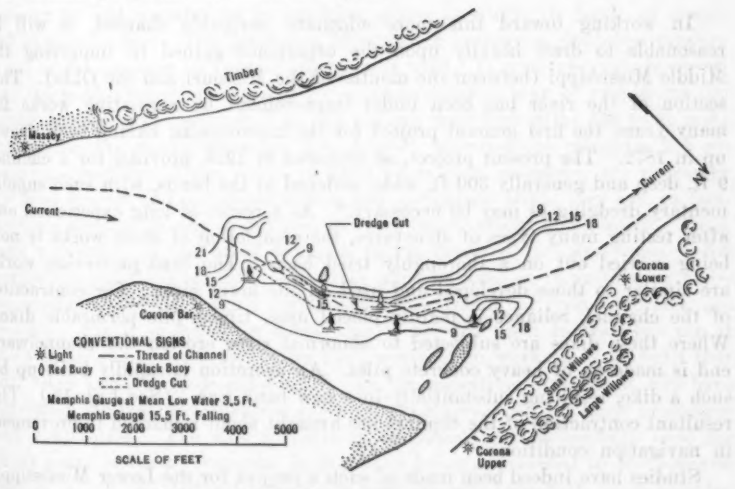


FIG. 10.—MAP OF MISSISSIPPI RIVER AT CORONA.

This need of additional channel width has already been recognized by the Commission, and approval has been given for a moderate increase, which

can be accomplished with the present dredging plant. The dredged cuts since early in the 1927 season have been not less than 300 ft. wide.

The question of additional depth is also important, although not pressing at present, inasmuch as the 9-ft. project depth coincides with that for the sections of river up stream. However, for a river with an average low-water depth of 30 ft. or more, a 9-ft. channel hardly seems a permanent solution from the viewpoint of ultimate economic development.

These comments are not intended to disparage the present method of channel maintenance by dredging, which has served navigation well, and which must continue for years to be its main reliance; but it is well to face frankly existing limitations. Mark Twain tells of a pilot, one Stephen, who when low of funds was induced to make a trip at a rate just half the regular stipend of $250 per month. The owner's satisfaction at his bargain was shaken when he observed Steve steering the boat up stream against the swiftest part of the current, taking the longer way around the bends, and allowing other boats to pass. His remonstrance was met by the reply that the other boats were handled by $250 pilots, and that he, Steve, knew as much as any man could afford to know for $125. With its dredged channel, the river has doubtless served navigation as well as a $600 000 river knew how; but after all, such a channel remains at best somewhat of a hand-to-mouth solution, a precarious foundation for a substantial commerce. Some day the growing needs of navigation are likely to demand a better solution.

IV.—As to Regulating the Lower Mississippi

In working toward this more adequate navigable channel, it will be reasonable to draw heavily upon the experience gained in improving the Middle Mississippi (between the mouths of the Missouri and the Ohio). This section of the river has been under improvement by regulating works for many years, the first general project for its improvement having been drawn up in 1872. The present project, as reviewed in 1926, provides for a channel 9 ft. deep and generally 300 ft. wide, widened at the bends, with such supplementary dredging as may be necessary.* As a result of long experience, and after testing many types of structures, the completion of these works is now being carried out on a thoroughly tried basis. The bank-protection works are similar to those developed and used on the lower river. For contraction of the channel, reliance is mainly placed upon timber-pile permeable dikes. Where these dikes are subjected to abnormal river erosion the channelward end is made up of heavy concrete piles. An accretion is rapidly built up by such a dike, resulting substantially in a new bank line. (See Fig. 11.) The resultant contraction of the channel has brought about a marked improvement in navigation conditions.

Studies have indeed been made of such a project for the Lower Mississippi, which is believed to be entirely feasible, given sufficient time and funds. This would involve a progressive treatment over a long period, regarding the

* See H. R. Doc. No. 9, 69th Cong., 2d Session, "Review of the Project for Improving the Middle Mississippi River."

river as a consecutive whole, shaping its banks and channel with infinite care and giving time for it to respond. A great advantage of the lower river in this respect lies in its large minimum discharge, and the moderate degree of amelioration required, compared with what might be attained (and which may some day be desired); that is, the degree of contraction required would be comparatively small.

The structures themselves would have to be made somewhat larger and sturdier, on account of the greater dimensions of the lower river. These structures would not be confined to any particular existing types, but would utilize the best types emerging from experimental development work. One promising type which has been used extensively on the Missouri River is the retard, similar to the Brownlow Weed of Indian rivers and the old abattis type on American streams. It consists generally of a series of spur dikes of built-up tree trunks deeply anchored by concrete piles. (See Fig. 12.) It is used both for bank protection and channel contraction, and is now being tested on the Mississippi.

On the Upper Mississippi, with an abundance of coarse sand and gravel readily available along the banks, it has been found that spur-dikes can be constructed effectively and cheaply of sand and gravel, with a capping of brush mattress weighed down with stone. Their success suggested the trial of this type of dike on the lower river, where on account of different soil and conditions it is of far more limited application.

Such a plan of regulation (which would be in fact a return to the Mississippi River Commission's early policy) would provide a channel distinctly superior to that maintained by dredging alone, in width, in depth, in shape, and in permanence. The great cost of complete regulation doubtless renders it out of the question in view of any present need. It is noteworthy, however, that the bank protection required for a comprehensive plan of regulating the Lower Mississippi is estimated to cost no less than four-fifths the total cost; the contraction works necessary for channel improvement comprise the remaining one-fifth. These works of bank protection, moreover, are necessary for flood control and soil conservation and navigation alike; they are, in fact, the basic element in any sound plan of river improvement and control. Thus, bank protection is the great common factor involved in improvements for navigation and flood control. The river must be fixed at its critical points—the caving bends—before any permanent works, including river terminals, are possible. Fix it in whatever shape that is decided upon as best, but pin it down somehow, somewhere. After that, contraction works may be built that will "stay put", if made substantial. In the meantime experiment will have developed the best and cheapest types of structures, both for bank protection and contraction. The willow mattress developed and successfully used for many years in bank revetment, is gradually being superseded by the concrete mat, of which two distinct types have been evolved, both with much promise. (See Fig. 13.) Other types of structures being tested for various purposes of bank protection or channel contraction include the retard and the sand-dike, both mentioned previously.

V.—Relationship of Improvements for Navigation to Flood Control

Navigation and Bank Protection.—As has already been suggested, it can be confidently expected that a complete system of bank protection, whether or not followed by contraction, will materially improve the navigability of the Lower Mississippi. This will be due partly to the exclusion of much of the enormous mass of eroded material from the caving banks, and partly to the greater stabilization of regimen, resulting from fixing the river in the concave bends.

Under the present program of operations, only about 6 miles of bank was protected by new work during the fiscal year of 1927. This is less than 2% of the total amount of bank that will need protection within the next decade. In view of the fundamental importance of this work from every viewpoint, it is reasonable to suppose that with more adequate funds the present program of operations will be largely expanded.

In this important and extensive program of bank stabilization, there will be need for the most careful planning to forward the interests of navigation as well as to meet the pressing demands of flood control. In the relatively small amount of bank protection that has been completed, much of it of an emergency nature, it has not always been possible to locate the work so as to serve these various purposes. In laying out a complete system, however, the opportunity surely will not be lost to utilize to the maximum the beneficient effect of bank revetment upon the regimen of the stream. This will mean that some banks will be permitted to wear away before protecting them, that others will be re-shaped to a flatter curvature, that, in short, all work done will be in conformity with, and contributory to, such a plan of complete regulation as may some day be warranted.

Influence of Floods on Navigable Channel.—It is noteworthy that the worst flood-years are also the most difficult years for navigation. Thus, the most troublesome seasons for the last ten years have been in 1922 and 1927, both flood-years. The obvious reason is that the abnormally high stages result in building up the bars to an unusual height (the rate of bar-building being often one-half as great as the rise of the river itself). When the river eventually falls, and especially if that fall be rapid, the river does not immediately "cut out" a new channel through the bars, but leaves them at higher than ordinary levels. In 1927 it was necessary to commence dredging operations at some crossings when the river stages were still 15 to 20 ft. above mean low water, or about 5 ft. higher than for the average year.

Moreover, the occurrence of crevasses naturally tends to cause deterioration in the channel below the crevasse. Instances of this are numerous, the most obvious evidence of this kind noted during the present year, 1927, being at James Bayou Crossing, about 10 miles below the Dorena crevasse. Whereas no dredging has been required over this entire reach for many years, already this season it has been necessary to dredge twice at this point. The more complete control and alleviation of floods would diminish such navigation difficulties.

Influence of Regulating Works upon Flood Heights.—It might be advanced, as an argument against regulation, that contraction works, being in

FIG. 11.—PERMEABLE DIKE AS USED ON THE MISSISSIPPI RIVER.

FIG. 12.—RETARD AS USED ON THE MISSOURI RIVER.

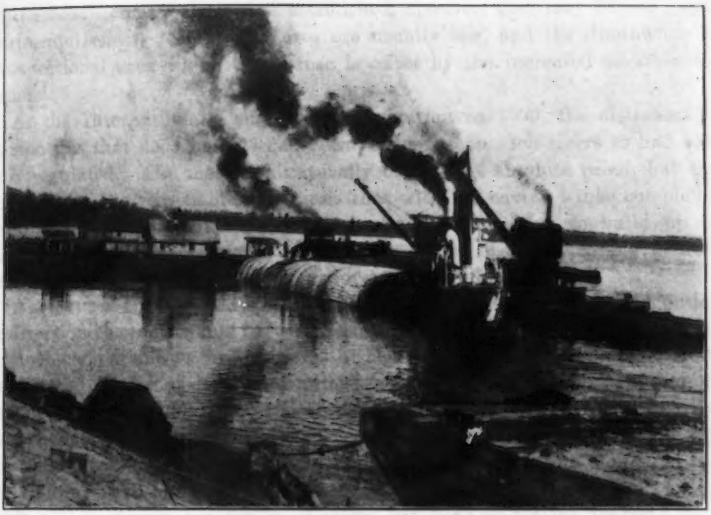


FIG. 13.—VIEW SHOWING THE LAYING OF AN ARTICULATED CONCRETE MATTRESS.

FIG. 14.—DREDGES ASSIST IN CLOSING THE MOUND CREVASSE.

one sense an obstruction in the bed of the river, would have the effect of raising flood heights. It is well established, however, that they have no such detrimental effect. Such structures are usually low, and the diminution in cross-sectional area which they cause is offset by the increased scour in the channel.

At the International Congress of Navigation in 1900, the consensus of opinion was that flood heights had been reduced upon such rivers as had been fully regulated. The matter is naturally difficult of absolute proof, but this accords with the reasonable hypothesis that with the caving banks completely protected, the rising river finds less material with which to build up its bars, and, therefore, maintains a better high-water channel, tending to draw off the floods more rapidly and to reduce flood heights.

Navigation and Spillways.—It is doubtless true that uncontrolled outlets or diversion channels would be very harmful in their effect upon the navigable channel, comparable to that of crevasses just mentioned. From this viewpoint the general theory as to the advantages of contracting the flow in a single channel is unassailable; but it is significant that the spillways or diversions proposed for the Lower Mississippi contemplate long overflow weirs, with their sills fixed at a level which will not affect low or medium stages, so that the spillways will operate only when relief is needed from excess flood waters. With such spillways, navigation, it seems, can have no serious quarrel.

Navigation and Reservoirs.—The potential capacity of reservoirs to serve navigation is well known, but the practical limitations as to cost and as to the difficulty of utilizing reservoirs for more than one purpose are also generally appreciated. Each case must stand on its own merits.

The studies as to possible reservoir effect now under way are being made primarily from the viewpoint of flood control, and navigation could probably expect only such incidental benefits therefrom as would be in the nature of a by-product. The writer has no data relative thereto, in addition to what have been given, except the statement that if all the reservoirs above Cairo thus far studied were built, they would increase the low-water flow below that point by an amount estimated at 150 000 sec-ft. for a period of 6 months, which would about double the low-water flow, and greatly diminish the dredging required. The great cost of such a system of reservoirs, however, would probably exceed any economic benefit to be derived therefrom.

Navigation in Relation to River Shortening.—In reviewing the report of the Special Committee of the Society on Floods and Flood Prevention,* the writer has been interested in noting the remarks by Messrs. Chittenden, Grunsky, and Townsend, as to shortening the Mississippi. The late General Chittenden thought that the river below Cairo might advantageously be shortened from 125 to 150 miles, or about one-eighth of its length; that the real difficulty in shortening lay in the damage to riparian interests rather than in the disturbance to the regimen of the river. More radical programs of shortening and straightening have also been proposed, some of which go so far as to visualize a mammoth straight ditch with concrete sides. The effect

* *Transactions*, Am. Soc. C. E., Vol. LXXXI (1917), p. 1218.

of "improving" in the sense of thus straightening a stream for drainage purposes, was also discussed in a paper presented by L. K. Sherman, M. Am. Soc. C. E., to the Western Society of Engineers, in 1923.*

With the effect of such a program of shortening on flood control this paper is not concerned. However, among the weighty objections advanced against shortening is that of interference with navigation. It appears to the writer that this danger has been exaggerated.

Thirty miles above Memphis is situated Island 35, which divides the river almost equally in two. Of late years the main current, which has long gone the more devious way around the bend, has been tending more and more to take the route through the chute, shorter by 4 or 5 miles. Early in the season of 1927, the Dredging District dredged as hitherto in the bend, with little success as to securing a channel which would remain open. Later, when that channel was plainly playing out, a dredged cut was made in the chute, supplemented by snagging operations. With some initial difficulty this channel has been maintained, and the boats are running there now with increased facility. The criticism that the river had been shortened by causing a cut-off was of course unjustified. The river itself had gradually selected and improved the chute way, and the dredging was merely a recognition at the proper time of that fact.

In this case navigation readily adapted itself to the mildly shortened channel. The pilots of the big river tows, in fact, usually prefer (where there is any choice) the shorter and straighter way; nor do the straighter reaches, on the whole, give greater maintenance difficulty than the crooked ones. A similar case in point is that of Racetrack Chute and Reid Bedford Bend, south of Vicksburg, Miss. The preferable channel, from the navigation viewpoint, is *via* Racetrack, the straight path through the chute, which is a mile or two shorter than around Reid Bedford Bend. It is worthy of note that the swiftest current in this vicinity is found in the point of the bend where the channel is quite narrow. It is not the average current that hinders navigation, but the swift current at some particular spot, as in climbing a reef or negotiating a sharp turn. The average slope below Cairo is less than two-thirds that for the St. Louis-Cairo Section, and, in the writer's opinion, both slope and average velocity might be somewhat increased without detriment to navigation, if attention were paid to the critical points.

Suppose, for example, that by encouraging every tendency of the main channel of the river to take the shorter way, as in the two cases mentioned, it proved feasible to gain as much as 1 mile in every 20 miles, or a total of 50 miles. Of course, other critical points would have to be strongly held to prevent a corresponding lengthening of the channel elsewhere; this should be feasible with an expanded program of bank revetment, with which program the shortening would go hand in hand. Such a moderate program of shortening would have better chance of success than a radical one, and would not require the same sweeping and difficult adjustments. Naturally, at flood stages

* "Experiments on the Effect of Upper Channel Improvements upon the Downstream Flood Heights," by L. K. Sherman, *Journal*, Western Soc. of Engrs., Vol. 28, 1923, p. 293.

the river would continue to use both channels, but a greater proportion of the discharge would go the shorter way.

The writer's point is not to contend that shortening is advantageous, but to suggest that the treatment of the navigable channel, which in general departs from the natural regimen as little as practicable, would probably adapt itself without detriment and perhaps with benefit to whatever moderate program of shortening might prove beneficial from a flood-control and drainage viewpoint.

Dredging and Flood Control.—In addition to their normal tasks connected with the navigable channel, the dredges of the Mississippi River Commission have served more directly the cause of flood control. During the past several years the cutter-head dredges have been occasionally useful in filling land-side pits and "blue-holes" (thus making levees more secure) and in constructing levees of sand, naturally on a somewhat flatter slope than the standard cross-section. The task of closing the great Mound crevasse is about one-half a dredging proposition, and the dredges there have now completed three-fourths of their work ahead of the original schedule and at a cost greatly below the bids received in response to advertising. (See Fig. 14.) Thus, for any program of levee enlargement or reconstruction, the plant designed to benefit navigation has proved its ability to serve as well the needs of flood control.

VI.—Conclusion

Improvement of navigation and flood control are quite different problems. The former is chiefly concerned with the stream at its lowest stages. The latter has to do with flood discharges thirty to forty times the minimum discharge, and a materially different channel. Flood control seeks to prevent the river from being a destructive enemy; navigation wants it improved in order to make it a useful friend.

Yet, despite these differences, navigation and flood control have much in common, and few if any conflicting interests. A comprehensive plan of bank stabilization is urgently needed for flood control and will greatly benefit the navigability of the river. In this and other ways, by finding a solution to this vast flood-control problem, it will be feasible at the same time to advance the cause of navigation on America's largest inland waterway.

THE FLOOD PROBLEM OF NEW ORLEANS, LOUISIANA

By Marcel Garsaud,* M. Am. Soc. C. E.

The flood problem of New Orleans, La., and of the Mississippi Valley below the mouth of Red River, is unique because of the special treatment to which it is susceptible. In order to have a comprehensive understanding of the situation one should be familiar with the topography of the city, and should have an intimate knowledge of the present status of New Orleans as a city, and of its commercial importance as a seaport.

Topography of New Orleans

The City of New Orleans is co-extensive with Orleans Parish, having an area of 196.2 sq. miles, of which 19.7 sq. miles are on the right bank of the Mississippi River. The densely populated and highly developed portion of the city has an area of about 40 sq. miles, and lies in a great crescent-shaped bend in the river. Lake Pontchartrain lies north of the city, the distance from lake to river ranging from 4½ to 7 miles. Lake Borgne, an arm of the Gulf of Mexico, lies to the east of the city. Fig. 15 shows the relation of New Orleans to the water about it. Several canals extend into the city from Lake Pontchartrain. The Inner-Harbor Navigation Canal, with a lock near the river end, connects the river with Lake Pontchartrain.

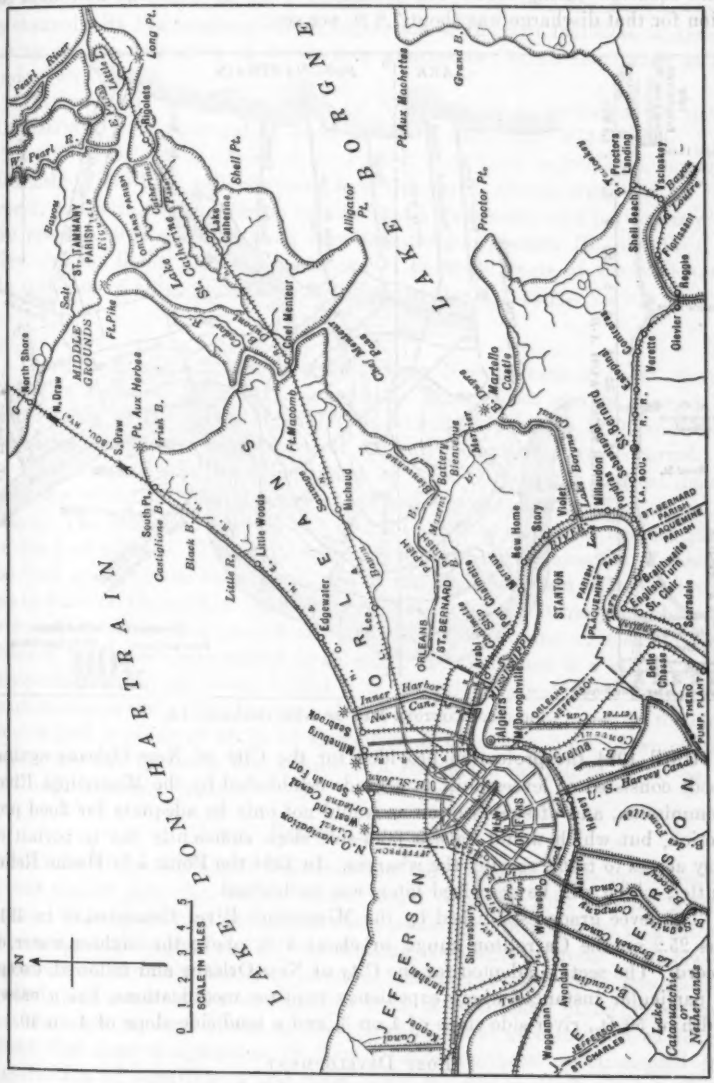
Except for a narrow strip of high ground along the river bank the ground surface in New Orleans varies in elevation from about 2 ft. below to 8 ft. above Gulf level. Fig. 16 shows the topography of the principal part of the city. (The contours are referred to Cairo Datum. The mean Gulf level is equal to 20.43 ft., Cairo Datum.)

The river at the Carrollton, La., gauge, located at the upper limit of New Orleans, fluctuates from slightly below, to about 21 ft. above, Gulf level. The range of fluctuation would have been increased by the 1927 flood had not the levee been breached at Caernarvon, a few miles below the city. The normal elevation of Lake Pontchartrain is about 0.3 ft. above mean Gulf level, but during windstorms it sometimes rises to a height 5 or 6 ft. above that level. It is necessary, therefore, that the city be protected against flooding from both the river and the lake.

Protection against the tides in Lake Pontchartrain presents no unusual difficulty. During the storm of 1915 the lake attained a record height of 6 ft. above Gulf level. Levees of very liberal section, with top elevations of 10 ft. above Gulf level, should provide ample protection against flooding from the lake.

The protection of the City of New Orleans against floods of the Mississippi River, however, presents an entirely different problem, toward the solution of which years of study and effort have been devoted and large sums of money have been spent. The river at New Orleans has a width of from 1 800 to

* Gen. Mgr., Board of Port Commrs., New Orleans, La.


LAKE PONCHARTRAIN
LAKE BORGNE
MISSISSIPPI RIVER
JEFFERSON
ORLEANS
ST. TAMMANY PARISH
Pearl River
R. Pearl R.
Long Pt.
Pt. Aux Herbes
Shell Pt.
Alligator Pt.
Pt. Aux Machettes
Grand B.
Proctor Pt.
Coal Mentour
Chef Menteur
Irish B.
South Pt.
Contigtione B.
Black B.
Little R.
Little Woods
Michaud
Bayou Bienvenue
Battery Bienvenue
Bayou Terre aux Boeufs
Bayou Bienvenue
Bayou Marcel
Bayou Villere
Bayou Dupre
Bayou Mazant
Bayou Gentilly
Bayou Sauvage
Bayou Chef Menteur
Bayou Paquebot
Bayou Lamarque
Bayou Trepagnier

3 400 ft. and a depth at high water of from 86 to 194 ft. The maximum discharge measured at Carrollton prior to the flood of 1927 was 1 358 000 sec-ft. and occurred during 1922. The velocity of the current at the minimum section for that discharge was about 7.8 ft. per sec.

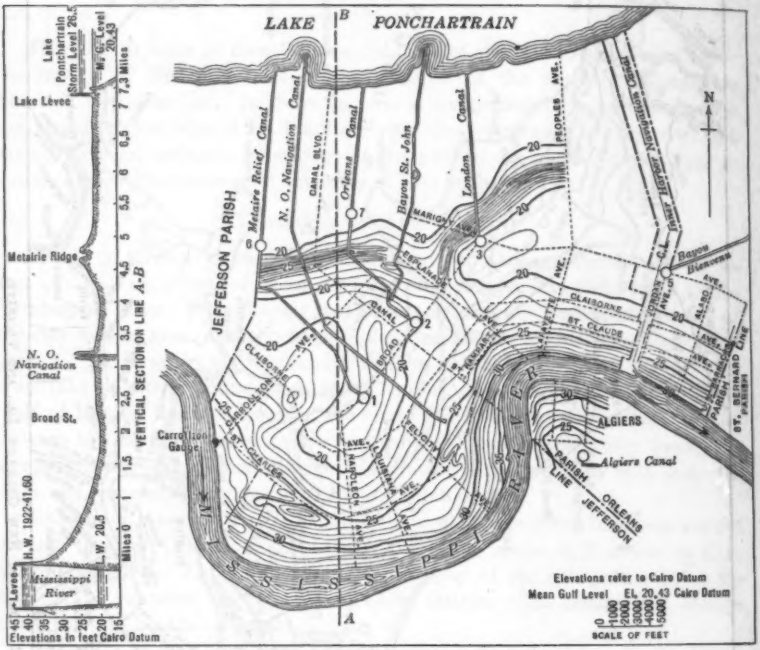


FIG. 16.—CONTOUR MAP OF NEW ORLEANS, LA.

Until 1924 the protection provided for the City of New Orleans against floods consisted of levees built to a grade established by the Mississippi River Commission, and of such section as would not only be adequate for flood protection, but which would have a land-side slope sufficiently flat to permit of easy access to traffic serving the wharves. In 1924 the Point à la Hache Relief Outlet, which will be described later, was authorized.

The levee grade established by the Mississippi River Commission in 1914 was 25.2 on the Carrollton gauge, or about 4 ft. above the highest water of record. The section adopted for the City of New Orleans and followed, except in particular instances where expediency requires modifications, has a crown width of 50 ft., river-side slope of 1 on 3, and a land-side slope of 1 on 10.

PORT DEVELOPMENT

The river banks of the City of New Orleans have been given over to commerce and are lined with modern, up-to-date wharves, docks, sheds, and warehouses. On the east bank of the river, of 12 miles of river bank, 7½ miles

are covered over by these structures. A great majority of these structures have been erected to conform to the Mississippi River Commission grade as established in 1914, and the State agency known as the Dock Board, which is charged with the construction and operation of the port facilities, is undertaking the reconstruction of its obsolete wharves to Commission grade as its budget will permit.

The value of the port facilities of New Orleans is estimated at about $100 000 000, with outstanding bond issues of more than $43 000 000. The Port of New Orleans is second to the Port of New York in foreign commerce. Steamship lines have sailings from New Orleans to almost every port of the world; and with the completion of the inland waterways, and particularly of the proposed 9-ft. channel from New Orleans to Chicago, Ill., the Port of New Orleans is destined to become one of the great ports of the world and the terminus of all the commerce and traffic of the Mississippi Valley.

Record Floods

The three greatest floods of record passing New Orleans in point of volume were those of 1882, 1912, and 1927. These floods can be classed as "super-floods". The floods of 1897, 1903, 1908, 1913, 1916, 1920, and 1922 were large floods; and many floods of ordinary magnitude have occurred.

Flood heights have been increasing at New Orleans for a great many years, due to the building of levees and the consequent closing off of the delta basins. The 1922 flood reached 21.3 at the Carrollton gauge, as against 14.95 for the flood of 1882. This quite naturally brought up the question of raising the levee grade on the lower river, and of the limit of height to which levees can be built on the soils in the vicinity of New Orleans. It has been observed in several localities, and particularly in the territory in and adjacent to New Orleans, that levees have settled or subsided, and that there is a certain point of equilibrium beyond which height it is not safe to construct a levee. An examination of the levees that have settled brought some engineers to the conclusion that a grade of 25 ft. at Carrollton is about the limit to which the levee grade can be set in this section if a reasonable margin of safety as to load limit is insisted upon.

Relief Spillways

A study of the 1922 flood indicated that much greater floods might occur. It was clearly seen that higher stages would occur at New Orleans in the future. This gave added impetus to the consideration of spillways as a means of lowering flood heights and thereby limiting the height of levees. The lowering effect of crevasses that had occurred during previous floods was closely studied, and a conclusion was reached by a large number of technical men that spillways in the vicinity of New Orleans would give the desired relief. About that time a committee of New Orleans engineers recommended the construction of a spillway a few miles below the city to discharge into Lake Borgne. Diversion of 250 000 sec-ft. was proposed. Due largely to the difference of opinion then obtaining among engineers regarding the effect of spillways, the recommendation was not carried into effect.

In 1924 the Louisiana Legislature passed an Act authorizing the Orleans Levee Board to construct a relief outlet on the left bank of the river below Point a la Hache, Louisiana. The writer was then Chief Engineer of the Orleans Levee District and was charged with the responsibility of reporting on this project. A favorable recommendation suggesting a justifiable expenditure of about $1 000 000 was made. It was estimated that by completely removing 11 miles of levee about 60 miles below New Orleans, flood heights would be lowered approximately 1 ft. at the Carrollton gauge.[8] The writer further stated that the discharge through the outlet at the crest of a flood like that of 1922 would exceed 250 000 sec-ft.

The Point a la Hache Spillway was constructed at a cost of $800 000; and observations taken during the flood of 1927 substantially confirmed the ideas of the writer as expressed in his report to the Orleans Levee Board. The curve in Fig. 17, indicates that the stage at Carrollton was 0.8 ft. lower than it would have been without the Point a la Hache outlet. The writer advocated the construction of this spillway not only to relieve flood heights at New Orleans, but also to provide an opportunity to make observations and to study the effect of spillways.

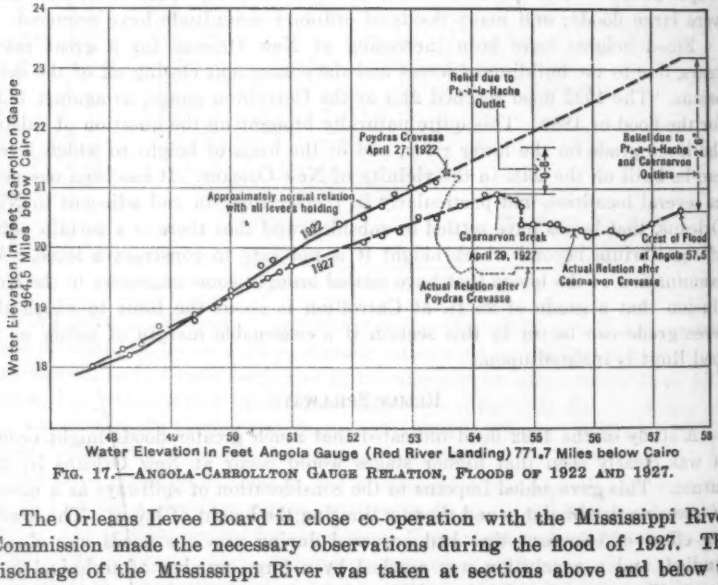


FIG. 17.—ANGOLA-CARROLLTON GAUGE RELATION, FLOODS OF 1922 AND 1927.

The Orleans Levee Board in close co-operation with the Mississippi River Commission made the necessary observations during the flood of 1927. The discharge of the Mississippi River was taken at sections above and below to determine the discharge through the gap. This work was done by the forces of the Mississippi River Commission. The Orleans Levee Board established gauges in the bays at the Gulf end of the outlet and observed them throughout the flood. All these data were furnished to the Federal Spillway Board.

About the middle of April, 1927, when the storms that produced the great flood were occurring, the U. S. Weather Bureau predicted a stage of 24 ft. at

the Carrollton gauge. This was nearly 3 ft. higher than any previous stage. The engineers and business men of New Orleans were confronted with an imminently dangerous condition. It was fully realized that, unless relief should come through a natural crevasse, such as occurred in 1922, emergency action would be necessary. As a result, the left bank levee was breached at Caernarvon about 22 miles down stream from the Carrollton gauge. The discharge through this gap was approximately 270 000 sec-ft. and the lowering effect at Carrollton about 2½ ft. The Caernarvon break not only gave effective protection to the City of New Orleans, but restored faith in the security of the city as well. In addition, it gave another opportunity to make observations and collect more data on mooted points. These observations were also furnished to the Federal Spillway Board.

Permanent Relief Measures

The writer has thus far attempted to sketch briefly the history of the flood question in the lower valley in order that the solution proposed by New Orleans may be more readily understood when presented. The flood problem of the lower valley consists of the necessity for flood-protection works of permanence, adequate not only to protect this part of the valley but also to preserve the integrity of New Orleans as a city and as a port. New Orleans has a population of more than 400 000 and property values of about $1 300 000 000.

It is estimated that the maximum rate of inflow at the latitude of Old River (mouth of Red River) during a flood like that of 1927 would be about 2 500 000 sec-ft. under confined conditions. As the Ohio River System was only in moderate flood at the time it is possible for a flood to occur that would be considerably larger than that of 1927. For this reason the greatest possible inflow is assumed at 3 000 000 sec-ft. at the latitude of Old River. The Lower Tensas Basin forms a reservoir just above Old River in which a large quantity of water is stored during floods. For this reason the maximum outflow below Old River may be taken as somewhat less than the inflow, or, say, 2 900 000 sec-ft.

Even to the casual engineer observer it is obvious that the Atchafalaya Basin is the logical place through which to discharge a large part of the excess flood waters. It is suggested, therefore, that this basin be converted into a floodway by removing the levees at its upper end and building side levees to control the flow throughout the full length of the basin. For the maximum assumed flood the Atchafalaya floodway would carry 1 100 000 sec-ft. A controlled spillway of 350 000 sec-ft. capacity would be built on the left bank at Bonnet Carre to discharge through Lake Pontchartrain. This would leave 1 450 000 sec-ft. to pass New Orleans.

Under present conditions, and with levees holding, this flow would produce a stage of not less than 22.5 ft. on the Carrollton gauge. Such a stage would not only endanger the city, but would seriously interfere with the commerce of the port. A third spillway should be built therefore at the Caernarvon site, a few miles below New Orleans. This spillway would be given a capacity

of 250 000 sec-ft. and would lower flood heights at New Orleans about 2½ ft. With such a system of spillways the stage at the Carrollton gauge could be held to 20 ft. under the extreme conditions assumed.

Operation of Proposed Spillways

The Atchafalaya floodway would be uncontrolled and would operate during all floods rising above bank-full stage. In addition to lowering flood heights at and below Old River, it would give a corresponding relief in the Lower Tensas Basin, and would afford a gradually diminishing relief in the main river for a considerable distance up stream.

The other two spillways would be of the controlled type and would be operated only when necessary to prevent unsafe or undesirable conditions. It would be necessary to raise the levees moderately as far down stream as College Point. However, no increase in levee heights would be necessary between College Point and the Passes. The free-board at New Orleans would be 5 ft. for the assumed maximum flood and somewhat more than 5 ft. for such large floods as have occurred in the past.

The spillway at the Caernarvon site will increase velocities along the New Orleans front by reason of the water passing the city at a lower elevation. Too great a diversion at this site might produce scour and caving, and for this reason the capacity of the spillway is limited to 250 000 sec-ft., and the spillway built as a controlled type so that the amount of diversion may be regulated at all times. Velocities of about 8.5 ft. per sec. occurred at the smallest sections of the channel through the city during the flood of 1927. Higher velocities should not be permitted in the future. Recent soundings taken along the harbor front show some scour, which is sufficient indication that greater velocities than those that occurred during the 1927 flood should never be permitted.

In constructing the spillways, auxiliary works of drainage, railway, and highway re-adjustments, and various other works would have to be done, of course, none of which presents unusual difficulties. Fig. 18 shows tentative details of a spillway suitable for both the Caernarvon and Bonnet Carre sites as designed by Roger B. McWhorter, M. Am. Soc. C. E., Special Engineer of the New Orleans Levee District.

Expense Involved

The probable cost of the flood protection project outlined would be about as follows:

Atchafalaya floodway	$50 000 000
Bonnet Carre spillway	12 000 000
Caernarvon spillway	9 000 000
Levee work between Point Breeze and College Point	35 000 000
Total	$106 000 000

Although the writer has not made estimates of the cost of raising levees on the lower river for a project that does not include spillways, it is believed that an expenditure of more than $100 000 000 would be required for all levee work below Point Breeze, exclusive of that in the City of New Orleans. An estimate has been made to determine the probable cost of raising the levees and the wharves in the city to conform to the levee grade necessary if spillways are not used. The estimated cost of this work is $40 000 000. The writer thinks it is clear that spillways are not only necessary and desirable, but that they will effect a large saving in cost as well.

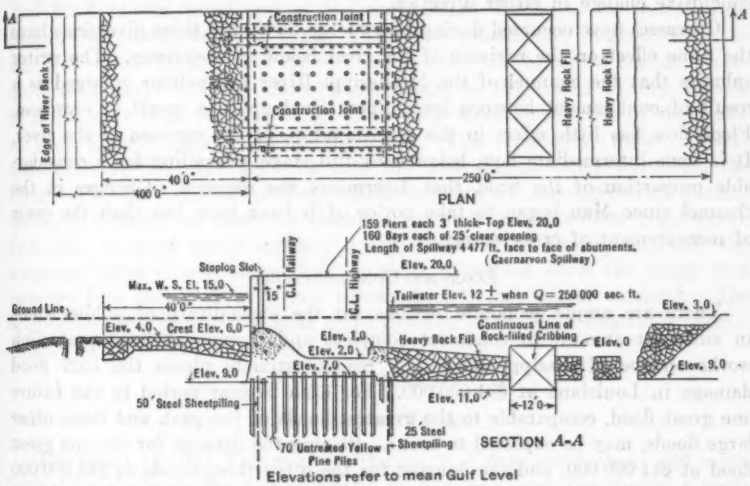


FIG. 18.—TENTATIVE SPILLWAY DESIGN, CAERNARVON SITE. SAME TYPE OF STRUCTURE SUITABLE FOR BONNET CARRE SITE.

The solution that has been given contemplates, of course, that the revetting of the river banks will not only continue, but will be carried forward at a more rapid rate in the future.

HISTORIC BASIS FOR SPILLWAYS

The writer has said little as yet in the line of argument for spillways, and little space will be devoted to the subject. To the best of his knowledge and belief there is not an engineer in the country who does not admit that spillways will effectively reduce flood heights. There are a few who believe the main river channel below a spillway will deteriorate as a result of silting. Fig. 17 plainly shows the effect produced by the Point a la Hache outlet and the Caernarvon break on the flood height at Carrollton. The Poydras crevasse of 1922, a mile or so up stream from Caernarvon, produced practically the same effect on the Carrollton gauge as the Caernarvon crevasse of 1927.

Originally, in its natural state, the channel of the Mississippi River was not nearly large enough to carry the flood flow and the excess water was stored temporarily in the delta basins and much of it carried over-bank down

the valley. Then, beginning at New Orleans in 1718, the levee system was built, most of this work having been done during the last fifty years. The delta basins were closed off and the river was confined between levees.

Those who favored "levees only" as a means of flood protection on the Lower Mississippi, as against levees supplemented by spillways, claimed that the confinement of the river would cause the channel to be scoured larger and deeper, and that flood heights would be lowered as a result; but such has not been the case. Cross-sections of the river have been taken from time to time since the Mississippi River Commission was formed, but they show no conclusive change in either direction.

Crevasses have occurred during every large flood, and these diversions have the same effect on the regimen of the river below as a spillway. The writer submits that the channel of the Mississippi River has neither enlarged as a result of confinement between levees, nor silted up as a result of crevasses. Flood flow has little effect in the determination of the regimen of the river. It is some intermediate flow, below bank-full stage, prevailing for a considerable proportion of the time, that determines the regimen. Changes in the channel since Man began to take notice of it have been less than the error of measurement of cross-section.

Economic Considerations

There are ample economic reasons for the expenditure of public funds in sufficient amount to construct complete and permanent flood protection works on the Mississippi River. A recent estimate places the 1927 flood damage in Louisiana at $81 000 000. For each 20-year period in the future one great flood, comparable to the greatest floods of the past, and three other large floods, may be expected to occur. Placing the damage for the one great flood at $81 000 000, and the damage for the other three floods at $15 000 000 each, the calculable loss in Louisiana during a 20-year period would be $126 000 000.

The loss to Louisiana, due to the fact that investment capital would certainly lose faith in the lower valley, would probably be several times that amount. There can be no doubt of ample justification for the expenditure of more than $100 000 000 for flood protection on the 300-mile stretch of river below Point Breeze. There is a broader view, however, than that based solely on economic returns. The flood problem of New Orleans and the lower valley involves the preservation of human life, as well as billions of dollars of property values.

The spillways suggested herein, especially the Atchafalaya Basin floodway, will restore to the river some of the characteristics it possessed when it flowed unmolested and unhampered in its natural state, prior to the time of levees which Man found necessary to build to reclaim valuable lands.

THE RELIEF OUTLETS AND BY-PASSES OF THE SACRAMENTO VALLEY FLOOD-CONTROL PROJECT

By C. E. Grunsky,* Past-President, Am. Soc. C. E.

Synopsis

The Sacramento River, throughout a large portion of its course from Red Bluff at the upper end of the Sacramento Valley to its outfall into Suisun Bay, flows between banks which are considerably higher than the adjacent valley land. The channel dimensions of the river are in places quite inadequate to carry the extreme floods, as illustrated by one stretch of river, more than 60 miles long, where only about one-eighth of the water at such a flood stage flows in the river. The remainder, under natural conditions, spilled over the bank into the flood basins which paralleled the river. The flood-control project, taking account of these physical characteristics, includes as essential features, not only relief outlets (spillways), but, also, selected areas between levees, so-called by-passes, to which the outgoing water is confined and in which it is carried down the valley to a re-entry into the river at points below which it has larger capacity. The application of such relief outlets and by-passes to the flood control in Sacramento Valley is here described.

Sacramento Valley

The Great Central Valley of California is a depression about 400 miles long, lying between the Coast Range on the west and the Sierra Nevada Mountains on the east. The northerly portion of this valley is known as Sacramento Valley. The northerly apex of the Sacramento Valley is at Red Bluff, at which point the valley begins at once to widen out, attaining and holding a width of about 30 to 40 miles throughout the lower half of its north and south extent. The Sacramento River enters the valley at its northern apex coursing down a little west of its center and discharging into the waters of San Francisco Bay. (See Fig. 19.)

In its meandering course through the valley the river has a total length of about 250 miles. Its fall from Red Bluff to the Bay is about 240 ft. at its low-water stage and a few feet more at its high-water stage, because the rise at flood is about 10 to 15 ft. more at the head of the valley than at points near the Bay.

The Sacramento Valley has a total surface extent of about 4 250 sq. miles (2 720 000 acres) of which the lands, that under natural conditions were subject to flooding and to occasional overflow during freshets, comprise about 1 000 000 acres, or nearly 40 per cent. Three-fourths of the lands which were thus subject to periodical and to occasional submersion by river and

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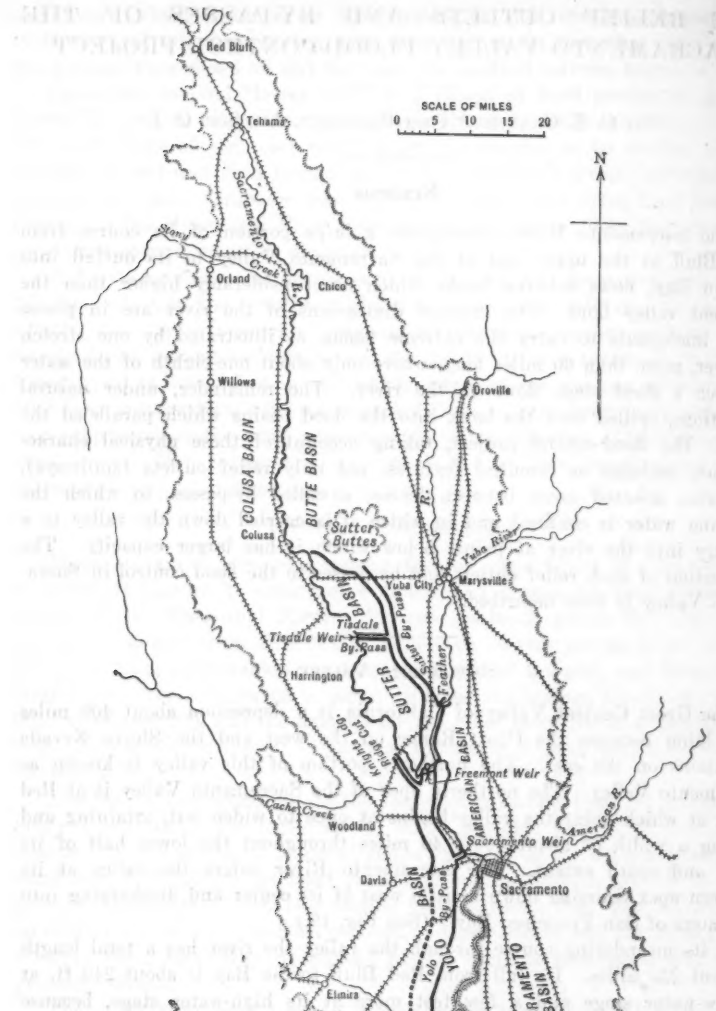

FIG. 19.—FLOOD-CONTROL PROJECT, SACRAMENTO VALLEY, SHOWING SACRAMENTO RIVER RELIEF OUTLETS AND THE BY-PASS SYSTEM.

freshet waters, lie in depressions or basins which flank the river. At times of high water there was an over-bank spill from the river into these basins and on a falling stream water returned therefrom to the stream at a number of points through natural drainage channels. The basins had thus become natural flood-flow retarding basins and had the effect of prolonging the high or flood stages of the river, thereby effectively and very materially holding down the river's peak discharge during flood conditions.

Sacramento River

This will be better understood from a brief description of the original condition of Sacramento River in its course through the Sacramento Valley. At the head of the valley the river has an apparent aggregate drainage area, as indicated by topographic features, of about 9 900 sq. miles. This area does not include about 1 500 sq. miles of territory in the northeastern portion of California and in Southern Oregon, to the northward of the river's visible drainage basin, from which there is supposed to be an underground flow southward, with discharge through springs into tributaries of the Sacramento River. However, it does include the secondary Goose Lake Drainage Basin with a water-shed of about 1 300 sq. miles, of which a portion is in Southern Oregon and the remainder in the extreme northeastern part of California. Pit River, as the main upper branch of the Sacramento River is called, received the occasional output of water from the Goose Lake Drainage Basin. This basin almost takes rank as an interior basin, because a long period has intervened since there was any outflow from Goose Lake into Pit River. The last notable outflow of which there is any definite record occurred in 1868. The rainfall in the water-shed of Sacramento River above Red Bluff varies from an average yearly minimum of about 15 in. on the northeastern plateau of California to upward of 60 in. in the vicinity of Mount Shasta. In high portions of the water-shed of Feather River, a Sierra Nevada tributary, the mean annual precipitation of rain and snow reaches 80 to 90 in. A large portion of the water-shed of the Sacramento River is composed of porous volcanic material into which rain and melting snows sink rapidly, feeding extensive natural and underground storage reservoirs from which there is at many points a large outflow in springs. These springs have a fairly constant flow throughout the year and, in consequence, are the principal source of the large low-water flow of the Sacramento River. This, under natural conditions, was about 4 000 sec-ft. at Red Bluff.

The flood stages of the Sacramento River result from winter rains. There is practically no precipitation from May to September, inclusive, which would produce material run-off. Rain in the winter months, however, and particularly a warm rain falling upon snow, may cause in a few days the river at Red Bluff to rise quite rapidly from very low stages to the highest. The ordinary winter flood stage at that point is about 22 ft. above low water. The peak discharge at an ordinary high stage of the river at Red Bluff is about 200 000 sec-ft.; at the river's extreme high stage its peak discharge may be as great as 270 000 sec-ft. When the river is in flood the extreme discharge at the head of Sacramento Valley does not persist for any great

length of time. The river ordinarily rises rapidly to maximum conditions and then its flow decreases in a comparatively short time to a moderate discharge. Over a period of a week or two the discharge at Red Bluff may be only one-fourth, or even less, of that at the extreme high stage of the river.

From the head of Sacramento Valley to Stony Creek, in a length of 58 miles, the river has an average fall of a little more than 2.5 ft. per mile. In this portion of the Sacramento Valley the slope of the land on both sides of the river is from the hills which encompass the valley toward the river. A number of small streams both from the east and west discharge their waters directly into the main stream. There are no extensive depressions or flood basins such as are found in the valley farther south. The channel capacity of the river is here sufficient to pass ordinary floods within the natural river banks, and the waters of larger floods which overtop portions of bank lands return to the river channel at points above Stony Creek.

This characteristic of the valley topography does not persist toward the south. The river in its farther course, as far as its delta, has built up high bank lands beyond which are the valley depressions, or so-called flood basins. The bottoms of these basins are here and there quite near to, but at other places several miles from, the river. The valley surface has ordinarily a quite uniform gentle slope toward the lowest points of these depressions, giving a smooth appearance to the valley floor.

The stream from Stony Creek to Butte Slough, a distance by river of 53 miles, flows on a lighter grade than in the more northerly portions of Sacramento Valley. Its fall in this stretch is about 1.3 ft. per mile, and the river cannot carry all the water which comes down in times of flood from the north. Some escapes over the banks into the depressions referred to previously. The river's original channel capacity in this stretch decreased gradually with distance down stream from about 150 000 sec-ft. to about 50 000 sec-ft.

From Butte Slough to Feather River, the Sacramento River has a very deficient capacity. Its gradient is here about 0.34 ft. per mile. Under natural conditions the river in this stretch could carry only about 20 000 sec-ft. All the remainder of the flood flow entering the valley from the north, east, and west, above Feather River, found its way into the flood basins and drained from them back into the river lower down. In this stretch the high bank lands are close to the river and there is a rapid fall, in places 15 to 20 ft. into the depressions on either side, making it impracticable to construct river levees far apart.

Below Feather River the Sacramento River in its original condition had a discharge capacity between 75 000 and 100 000 sec-ft., depending on the unknown channel dimensions now modified materially by human agencies, notably by the deposit in the river channel of great quantities of sand which originated in large part in the mountains in the days of hydraulic mining activity. The river gradient from Feather River to the first large partition of water in the river delta, at the head of Steamboat Slough, a distance of 48 miles, is

about 0.30 ft. per mile and somewhat less thence for 18 miles to the point where the water from the great lower west side depression, Yolo Basin, returns to the river and where the capacity of the original river channel increased to upward of 200 000 sec-ft.

Sacramento Valley Flood Basins

For an understanding of the flood-control project on this river it is necessary to say a few words about the Sacramento Valley flood basins which under natural conditions were very effective agents in decreasing the peak of the river's flood discharge. These basins are here very briefly described as they were and as they functioned before the flow of the river had been materially modified by human activities. (See Fig. 18.)

On the east side, in their order down stream, are Butte Basin, Sutter Basin, American Basin, and Sacramento Basin. On the west side are the Colusa Basin (frequently described as consisting of the Upper and the Lower Colusa Basins) and Yolo Basin.

Butte Basin, the uppermost east side depression, extends down stream as far as the contraction between the river's high bank lands on the west and the Sutter Buttes on the east. At this contraction, Butte Slough, a break through river bank lands, forms an interconnection between the river and the basin. The water which flows over the east bank of the river above Butte Slough flowing in a network of channels and uniting with water from a number of streams which drain outlying portions of the Sierra Nevada region, reaches and fills the Butte Basin, the drainage from which, during river flood stages, goes south through the contracted low area between the river and Sutter Buttes already referred to. During a general flood stage of the river this basin holds a slow moving sea of water, 30 to 150 sq. miles in area, depending on the magnitude and origin of the flood waters. The volumetric contents of the basin cannot be given with precision because in times of flood the surface of the water in the basin has more or less slope depending upon the source from which it receives its greatest accession of water.

The second flood basin on the east side of the river is Sutter Basin. This basin has a length from north to south of about 30 miles and an average width of 6 miles. The upper 10 miles thereof, however, has so much surface slope and lies so much higher than the more southerly portion, that, as soon as the inflow of water from the north ceases, there is rapid drainage into the more depressed southerly portion, and submersion of basin lands does not there continue long. The unwatering of the more southerly basin lands being by drainage into Sacramento River near its confluence with the Feather is a slow process. When the flood stage at the mouth of Feather River is at Elevation 30 (above mean sea level), and the basin is full to this height (which is about 8 ft. below the flood-control project high-water stage), its surface has an area of nearly 140 sq. miles and its contents range from 25 000 000 000 to nearly 40 000 000 000 cu. ft., according to the momentary volume of inflow from the north. When, for example, in December, 1889, at its highest stage of the winter, the water in Sutter Basin amounted to

39 000 000 000 cu. ft., or four times the quantity which would fill the Sacramento River channel below Iron Canyon from a low-water to a high-water plane. The general elevation of the lowest portion of this basin is 19 to 20 ft. This is but little above the elevation of the low-water plane of Sacramento River at the mouth of Feather River. Complete drainage of Sutter Basin as it was originally, was therefore a slow process.

The river does not fall to its seasonal lowest stage until in the autumn months, and drain-ways to it which Nature had provided, did not everywhere connect with the lowest spots. Drainage, therefore, was imperfect and water stood in some portions of the flood basin throughout the entire year. It is to be noted that by reason of its position between two rivers, this basin, before its reclamation, received water not only from the Sacramento, but also from the Feather River. While being filled and while discharging its contents, it had a pronounced effect on the flow of the Sacramento River below the Feather.

American Basin is the third east side basin. Its southerly or down-stream end is at the American River. Its northerly apex extends along the east side of Feather River for some miles above its mouth. The basin has a surface extent, if measured by such a flood as that of December, 1889, of 110 sq. miles and, at that time, at its highest stage it contained about 25 000 000 000 cu. ft. of water. Its water stage at its southerly end was controlled by the stage of Sacramento River at the mouth of the American. It is interesting to note that a change of 1 ft. in its elevation at a full stage represents as much water as goes by Sacramento City at a flood stage of Sacramento River in 10 hours.

Sacramento Basin is a name sometimes applied to the depressed area on the east side of the Sacramento River below Sacramento City. This depression is long and narrow. It has been so well protected by levees—although there have been occasional floodings—that its having ever had any material effect on the river discharge is generally ignored. About 35 sq. miles of the lowest portion of this depression are only 6 to 7 ft. above mean sea level, and, therefore, about 10 to 15 ft. lower than the original high-water stages in the Sacramento River opposite the upper end of the basin.

Colusa Basin lies on the west side of Sacramento River, being separated from the Yolo Basin by a ridge of relatively high ground built out from the Coast Range to Sacramento River at Knights Landing (Grafton), by Cache Creek. This basin is long and comparatively narrow. Before the water was held back by levees, any general river flood stage converted the entire west side valley trough from a point in the latitude of Princeton to the ridge at Knights Landing into an inland sea nearly 50 miles long and 2 to 7 miles wide. Its water came from the west side over-bank flow of the river and from numerous small creeks which descend from the Coast Range. During flood conditions, when this basin was full, and water was flowing in it from north to south, its contents may at times have reached 45 000 000 000 cu. ft.

Yolo Basin is the largest of the several flood basins along the Sacramento River. It extends from the ridge described at Knights Landing to Cache

Slough at the lower end of Grand Island on the south. It has a length of more than 40 miles and an average width of 7 miles. At times of general inundation when water was flowing down the basin, its volumetric capacity, before being reduced by reclamation works, was about 50 000 000 000 cu. ft. Its surface extent may be noted at nearly 300 sq. miles. During the high water of 1889 (this being a flood for which the records permit approximation), the discharge from the lower end of this basin back into Sacramento River through Cache Slough, its outlet channel, was more than twice the quantity of water which the river was carrying. In other words the natural capacity of the river at and below Sacramento was only one-third of the combined flow in the river and in the Yolo Basin and yet this combined flow was not as great as the discharge would have been, at the peak, if there had been no retarding influence by flood basins.

The Flood Control Project*

In view of the situation on the Sacramento River in the Sacramento Valley, as explained, the adopted project was based on the following basic principles:

1.—The river must be forced to carry the maximum amount of the flood flow which can be held between properly placed levees of reasonable height.

2.—The water presented at any time in excess of maximum river capacity must be allowed to escape from the river at selected points under control and only for such length of time as is necessary to keep the river below an adopted flood-plane elevation.

3.—The flooding of lands by the water which is allowed to escape from the river must be restricted to definite areas set apart for the purpose.

In the application of these principles to the Sacramento River, the engineers who advised the State and Federal authorities found it desirable to use river discharges as determined from the flood conditions of March, 1907,* as a general guide and to plan control works with some allowance for safety under similar conditions. Studies made in great detail under the direction of Capt. (now Col.) T. H. Jackson, Corps of Engineers, U. S. A., by the Civilian Assistant, H. H. Wadsworth, M. Am. Soc. C. E., and further amplified by E. A. Bailey, Flood Control Engineer of the California State Reclamation Board, led to an adoption of channel, relief outlet, and by-pass capacities in the main, as follows:

From the head of the valley to a relief outlet weir constructed in 1891, and known as Tisdale Weir, the channel capacity will decrease from about 260 000 sec-ft. to about 72 000 sec-ft. The excess over what the river just above Tisdale Weir can carry goes over the east bank of the river flowing as already described into Butte Basin and thence into Sutter Basin. Tisdale Weir has a fixed crest 1 100 ft. long, over which the discharge at flood stage is to be 38 500 sec-ft. (See Fig. 20.)

* *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), pp. 1488–1495.

Below the Tisdale Weir the river is to have a capacity of 33 500 sec-ft. between high levees and the remainder of the flood flow of this part of Sacramento Valley, estimated at 216 500 sec-ft. for a flood such as that of 1907, will be carried in the Sutter By-Pass. To this there will be added the flow of Feather River, 200 000 sec-ft., some miles above the lower or south end of Sutter Basin.

The total flood flow from the north, which will reach the vicinity of the confluence of Feather and Sacramento Rivers, is estimated at 450 000 sec-ft. being the 33 500 sec-ft. in Sacramento River and 416 500 sec-ft. by-pass flow, including Feather River. Of this quantity the Lower Sacramento is expected to take away 107 000 sec-ft., leaving 343 000 sec-ft. to be otherwise cared for. As there is no room for this in the lower river channel, it must be allowed to escape into Yolo Basin. For this purpose a relief outlet controlled by the Fremont Weir has been constructed on the right, or west, bank of Sacramento River a few miles up stream from the confluence of the two rivers. (See Figs. 21 and 22.)

At the City of Sacramento, 20 miles below the mouth of Feather River, the Sacramento receives its last tributary, the American River. It is estimated that this river at a general flood stage similar to that of 1907, will discharge 128 000 sec-ft. into the Sacramento River. This discharge alone, without regard to the water already in Sacramento River, would overtax the contemplated river capacity (110 000 sec-ft.) below Sacramento. To meet this situation a relief outlet has been constructed near the mouth of American River. It is known as Sacramento Weir. (See Figs. 23 and 24.) For economic reasons this has been placed about two miles up stream. Under this arrangement there will be, at times, an up-river current from American River to the relief outlet, but this will be so rare and will continue for such a short time that no bad effects, such as channel obstruction by silt, are feared. It is anticipated that under recurrence of a 1907 flood considerably more than 100 000 sec-ft. of water would go out of the Sacramento through this relief outlet.

At each relief outlet there is a permanent structure replacing a section of levee, through or over which the outgoing water flows. There is thus a Tisdale Weir on the left or east side bank of Sacramento River at the upper end of Sutter Basin; a Fremont Weir on the west bank of the river a few miles above the mouth of Feather River; and a Sacramento Weir also on the west bank of Sacramento River 2 miles above its mouth. The crest of the Tisdale Weir is at an elevation about 2 ft. above the general level of bank lands and about 9 ft. below the extreme flood height in Sacramento River. Whenever water reaches the crest height of the weir the outflow from the river begins and continues during flood stages until the water in the river again drops to below the height of the crest of the weir.

The Fremont Weir also has a fixed crest which is at the general level of bank lands. A trench was excavated to receive this structure. The length of its crest is 9 000 ft. Here, as in the case of the Tisdale Weir, flood water flows freely over the weir whenever the flood stage is higher than the crest. When

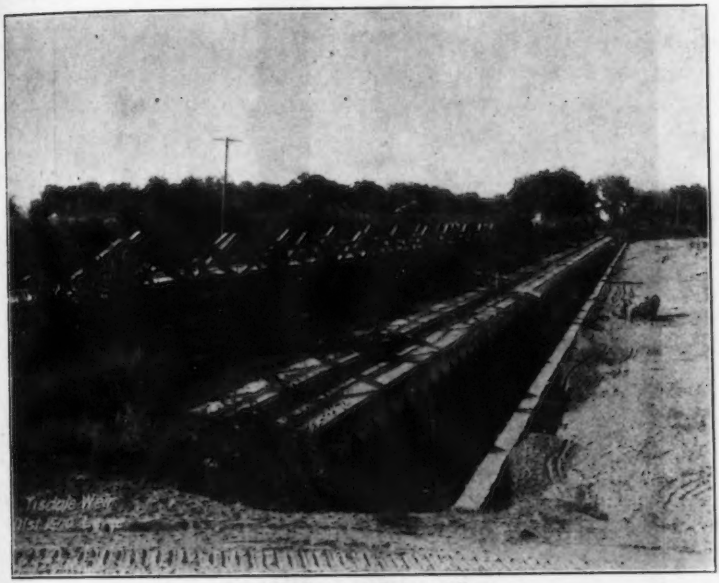

FIG. 20.—VIEW OF TISDALE WEIR.

FIG. 21.—VIEW OF FREMONT WEIR.

FIG. 22.—FREMONT WEIR, FEBRUARY 15, 1925.

flood conditions are similar to those of 1907 the depth of water on the weir will be about 7.5 ft.

The Sacramento Weir has a length of 2 000 ft. and is provided with a movable crest. The top of the fixed structure is about 10 ft. below the river flood stage and the movable crest adds 6 ft. to the effective height of the weir.

At all three weirs the force of the over-falling water is broken by a stilling device, a box-like depression constructed of concrete. In the case of the Fremont Weir, on account of the slight elevation of the crest above the land over which the weir discharges, the stilling-box is shallow and a down-stream apron of rock has been added for additional protection.

The water which flows from the river, through the various relief outlets is not allowed to spread out at random over the basin lands. (See Figs. 25 and 26.) It is confined to comparatively narrow channels by levees. The water which reaches Sutter Basin through the Tisdale relief outlet, together with the water that flows out of Butte Basin, is thus forced by levees, 4 000 ft. apart, over toward the east or Feather River side of Sutter Basin (see Fig. 25) and, as already stated, unites with the water of Feather River a few miles above the mouth of that river. By means of this arrangement the river's over-bank waters are carried around the main portion of Sutter Basin and about 55 000 acres thereof are afforded protection against the floods of both the Sacramento and the Feather Rivers. The water which flows through the Fremont relief outlet (Fig. 22), and that which flows through the Sacramento relief outlet, is likewise confined between levees, making it practical to prevent inundation of a strip of land on the west bank of the river and of considerable areas along the western margin of Yolo Basin.

It would go beyond the purpose of this paper to describe in detail the various areas which have thus received protection against river floods, or to discuss the economic aspects of the Sacramento River flood-control problem. It will suffice to add to the foregoing description that the location of the various by-pass areas necessitated the construction of some quite high levees. Thus, for example, in the case of the Sutter By-Pass there are long stretches of levee from 20 to 25 ft. in height. At times of flood the water of the by-pass stands against these levees for days at a time and subjects them to the usual danger in such circumstances of being attacked by wave action or threatened by seepage waters.

The by-passes have considerable width. The one through the Sutter Basin, as noted, is 4 000 ft. and more in width. That through Yolo Basin ranges from 7 000 to 14 000 ft. When they are filled with water this great width gives the wind a free sweep and subjects the by-pass levees to attack at exposed points by wave action, making their protection with facing of rip-rap or concrete at exposed points necessary.

The fact that high levees are vulnerable is generally recognized. This is particularly true where as in the case of the California by-pass levees there is a long period each year in which there is no rainfall, and in which, therefore, the levees become very dry and subject to cracking where clay enters into their composition. Absolute assurance of protection behind high levees can-

not, therefore, be given. (See Fig. 27.) When this is taken into account, together with the additional fact that volumes of water to be handled at the extreme flood cannot be known, because any flood stage of the past may be surpassed in the matter of volume of flow sometime in the future, then it appears quite questionable whether attempts to protect completely such areas as Sutter Basin should ever be made. This basin might well have been acquired by the State of California and held for all time as an overflow recipient functioning as a retarding basin. Had this been done much of it would have been free of overflow in May or early in June of most years and could have been farmed to summer crops. There would have been little or no economic loss to the State. Furthermore, the flow into this basin and the discharge from it would have prolonged the duration of floods and would have cut down the peak discharge of the lower river, thereby simplifying and rendering safer the protection works on the lower river.

The Sacramento Valley flood-control works have thus far functioned under moderate tests only. There has been no flood stage comparable with that of 1907, nor even with that of 1909, since the completion of the two lower relief outlets. There has, however, been water in the by-passes, notably in February, 1927, when it reached an elevation 10 ft. below the levee tops in the Sutter By-Pass. All that can be said at this time of the relief outlets on the Sacramento and of the by-passes is that their introduction as features of the flood-control works was based on sound principles, but that it is still questionable as to whether these principles have been throughout properly applied to the existing situation.

That the flood-control problem in the Mississippi Valley is comparable with that in the Sacramento Valley seems to be established by recent events. When such volumes of water are presented as was the case during the 1927 floods in the Mississippi Valley, the peak discharge would demand a waterway far in excess of what can everywhere be provided between river levees of reasonable height. Moreover, no one can give assurance that some time in the future there may not be even a worse combination of discharges from tributaries and still greater demand for capacity. It is apparent, therefore, that the present program of levee building should be approved and accepted as a first step in the execution of a comprehensive project, to be, however, supplemented with relief outlets—always preferably in the nature of spillways with a high crest level—through which the river can unload its surplus water.

At such relief outlets no water should go out except when the river rises above a predetermined danger stage, and the discharge through them should cease as soon as the river is again below the danger line. The outgoing volume of water and the flooding of lands will thus be kept at a minimum as will also the number of days or hours during which the outflow continues. Moreover, the areas that will be flooded should be selected in advance, as in the case of the Sacramento River by-pass areas, and they should be held in public ownership in order that farming activities thereon can be kept under control and erection of improvements that would be too greatly damaged by occasional floods can be prevented. It will readily be seen that under such

FIG. 23.—VIEW OF SACRAMENTO WEIR, FORTY-FOUR GATES OPEN.

FIG. 24.—VIEW OF SACRAMENTO WEIR, SHOWING CLEARING AND GRADING BELOW WEIR.

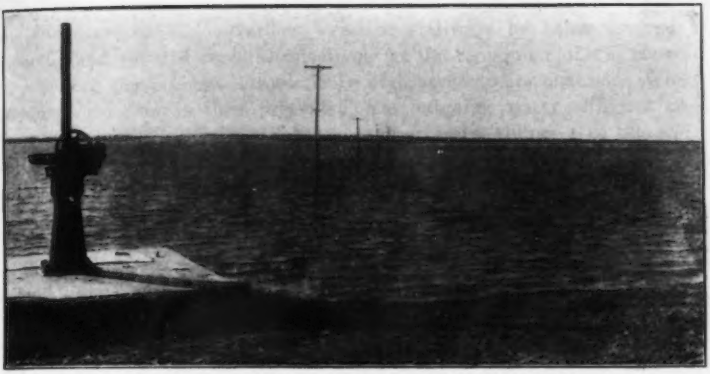

FIG. 25.—SUTTER BY-PASS OPPOSITE TISDALE BY-PASS.

FIG. 26.—VIEW OF SACRAMENTO BY-PASS, FEBRUARY 6, 1925.

FIG. 27.—VIEW OF SACRAMENTO RIVER, SHOWING DOLAN BREAK IN WEST LEVEE, BELOW COLUSA, CALIF., FEBRUARY 4, 1915.

a plan great stretches of levee, perhaps miles in length, can be held relatively low and converted into overflow weirs or spillways by being covered with concrete, and yet will serve as effectively as the remainder of the river levee in confining the river at a reasonably high stage to its channel. When an extreme flood flow is then presented, the outgoing water will not be discharged through deep accidental breaches in excessive volume (see Fig. 27), but will go out at selected points over high barriers and will be held at the minimum amount which will afford the desired relief.

Cost of the Project

This project is not merely for flood control. The lower end of Sacramento River below Cache Slough is being widened and deepened and navigation conditions there and elsewhere along the river are being protected and improved. The cost of the project falls upon the United States, the State of California, and the land owners, as indicated by the following figures taken from the report of the State Engineer under date of February 10, 1925:

Total Federal appropriations to June 30, 1924		$3 749 900
State Appropriations:		
Co-operative work	$3 500 000	
Lands, cut-off, and channel enlargement	248 600	
For Sutter By-Pass	3 000 000	
Total by State		6 998 600
Land owners' expenditures and obligations		20 497 804
Total appropriations, expenditures, and obligations		$31 246 304
Estimated cost to complete		20 000 000
Estimated total project cost (since 1910)		$51 246 304
Expended by land owners on works useful in the project prior to 1910		$7 272 526
Expended by land owners within the area protected by this project, on back levees, drainage, and other works necessary for land reclamation, since 1850 to December 31, 1924		$41 344 445
The cost of levying assessments from time to time, interest, discounts, and other costs of financing, is estimated by the State Engineer at		$7 657 148
The land owners' expenditures for maintenance of flood control and reclamation works to December 31, 1924, are estimated by the State Engineer at		$8 561 643

Infrequency of Extreme Flood Flow

It is now generally recognized that records of precipitation do not cover long enough time periods to show with positiveness any tendency toward permanent change in the amount thereof or in its intensity. Moreover, the combination of circumstances which produce extreme flood conditions are not of frequent occurrence. Abundant evidence can be found to show that greater floods than any of which there is record are still occurring from time to time and that the possibility of such greater floods should not be lost sight of when flood-control works are being planned.

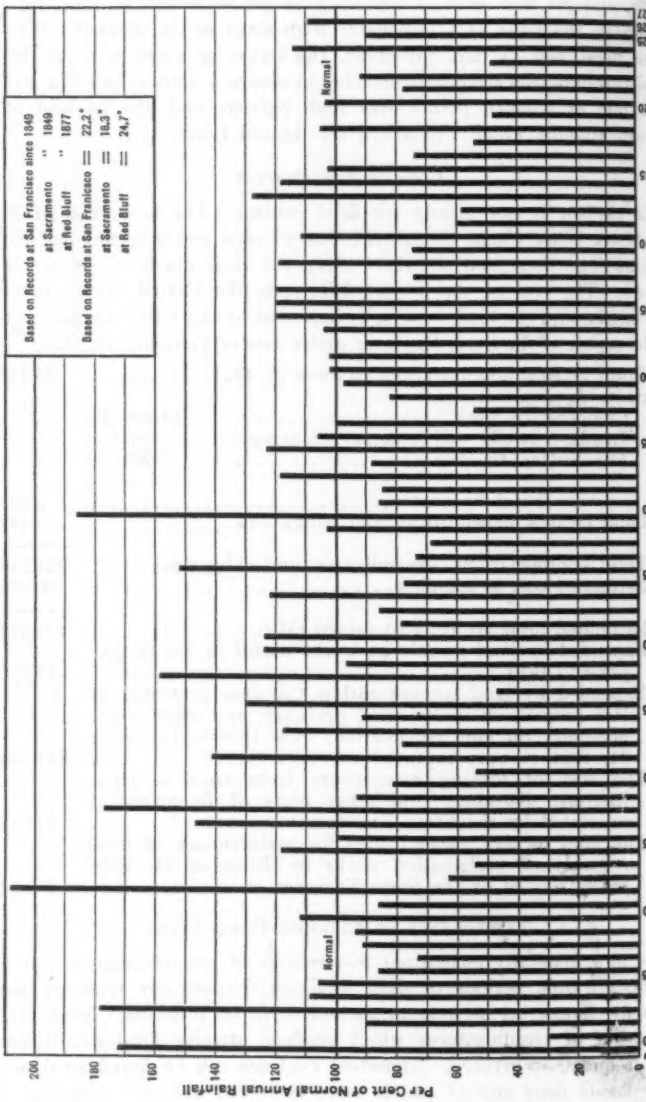

FIG. 28.—RAINFALL IN CENTRAL CALIFORNIA, 1849 TO 1926.

The following facts are cited by way of illustration:

The City of Szegedin, Hungary, on a tributary of the Danube, had been protected by levees for about 100 years when, in 1879, water overtopped the levees and great disaster resulted.

Salt Lake, Utah, which has no outlet, was low in 1843.* It was high 20 to 30 years later, then fell to a low stage (culminating about 1896) comparable with that of the Forties. The claim was then freely made that consumption of water from tributary streams for irrigation had been the cause of the lowering and would keep the lake low. Despite such depletion the lake rose again to a high stage and has remained comparatively high since 1908.

Tulare Lake, California, according to Indian tradition, was once dry. It was certainly very low for a long period preceding the arrival of the white man. This is evidenced by tree stumps uncovered about 1881 by its receding waters. It was filled to overflowing in 1861-62; again to its highest stage in 1867-68. It has been dry for a number of years in recent times, but will surely be temporarily restored to a sizeable area sometime in the future.

On the Danube, in 1897 and 1899, high waters at Vienna rose to within an inch or two of the levee crest; and, yet the river discharge at the peak of the flood conditions was only about three-fourths of the discharge, estimated from high-water marks, of the flood of 1501.

In California, the last winter of the type most likely to produce extreme flood stages in the Sacramento River was that of 1889-90. (See Fig. 28.) Similar winters, earlier in the history of the State were those of 1849-50, 1852-53, 1861-62, and 1867-68. Their absence since 1890 gives no assurance that there has been a change of climatic conditions. Such winters will occur again. No one can tell how soon.

It is possible, and should be regarded as probable, that floods in such winters will surpass those of 1907, 1909, 1911, and 1914, which have served as recent object lessons to the flood-control engineers of the State.

* See Fremont's Reports.

RECLAMATION AS AFFECTING FLOOD CONTROL

By Elwood Mead,* M. Am. Soc. C. E.

According to the 1920 Census figures, there are in the United States a total of 7 538 irrigation reservoirs with an aggregate storage capacity of 21 246 436 acre-ft. Of these, 1 601 reservoirs with an aggregate capacity of 6 023 922 acre-ft. are located within the Mississippi River Drainage Basin. All these reservoirs would be filled with about 4½ days' average discharge of 1 375 000 acre-ft. per day, and amounts to less than 2 days' flow of the Mississippi at maximum flood, maximum discharge being estimated at 1 806 000 sec-ft. It is apparent, therefore, that the flood-reducing effect of the irrigation reservoirs now in existence is negligible.

Approaching the problem from another angle, the total irrigated area in the Mississippi Basin (1920 Census figures) is about 5 000 000 acres. Assuming an average diversion of at least 2 acre-ft. per acre, there would be roughly 10 000 000 acre-ft. by which the discharge of the river during the irrigation season has been reduced by reclamation development; and this reduction is effected during the period of the great Mississippi floods, as the return flow arising from the irrigation diversion dam does not reach the stream until after the flood period.

According to the same Census figures, the total irrigable area included in existing irrigation enterprises in the Mississippi Basin is about 10 000 000 acres, and there is, therefore, an apparent opportunity by the extension of these projects to reduce the flow of the river by another 10 000 000 acre-ft. Assuming that this is spread out uniformly over an irrigation season of 180 days, it would effect an average reduction in flow amounting to 28 000 sec-ft.; or about 4% of the average flow of the stream.

On the Mississippi itself, the serious floods arise elsewhere than in the irrigated regions. The effect of reclamation on the control of its floods appears, therefore, to be negligible; its effect on those tributaries lying within the arid regions is, however, a controlling factor. On the North Platte River former destructive floods have been practically eliminated by the construction by the U. S. Bureau of Reclamation of the Pathfinder and Guernsey Reservoirs. These two reservoirs provide 1 140 000 acre-ft. out of the total storage on this stream of 1 800 000 acre-ft. On the South Platte, 338 reservoirs are in operation with a storage capacity of nearly 1 000 000 acre-ft. On the Arkansas, 367 reservoirs provide 1 155 000 acre-ft. of storage, and the effect within the valleys of these streams, as well as other tributaries on which storage has been provided, has been highly beneficial.

Farther west are regions where flood protection by irrigation reservoirs has been most marked. One of the U. S. Bureau of Reclamation reservoirs, namely, the Elephant Butte, on the Rio Grande, was designed with the purpose of setting aside a definite portion of its capacity for flood control. Its total

* Commr. of Reclamation, U. S. Dept. of the Interior, Washington, D. C.

capacity is 2 638 000 acre-ft., of which the upper 400 000 acre-ft. is reserved for this purpose. The rapidly alternating conditions of extreme shortage and devastating flood characteristic of many streams in the arid region, are graphically portrayed in the following excerpt from a report made in 1921 by L. M. Lawson, M. Am. Soc. C. E., at that time Superintendent of the Rio Grande Project:

"In the relatively short time of five years since the completion of the Elephant Butte Dam, it has served the two extreme requirements, namely, that of storage and that of flood control. The crop produced during the period of 1917 and 1918, when practically all the water used for irrigation was drawn from storage, had a value in excess of the cost of the dam. The flood damage which would have resulted from the uncontrolled discharge of the flood of 1920, through the valleys of the Rio Grande above and below El Paso, would have reached an amount also largely in excess of the cost of the structure."

On the other hand, the fact that flood protection cannot in every case be expected to be provided by the reservoirs necessary for irrigation, is demonstrated by conditions on the Middle Rio Grande Conservancy District. This District comprises lands in the Rio Grande Valley between the upper end of the Elephant Butte Reservoir and the Colorado-New Mexico State line, and has been organized in order to determine in a proper and co-ordinated manner the particularly acute problems of flood control, drainage, and irrigation, which must be solved to permit the fullest development of that part of the Rio Grande Valley. Here, the engineers have reached the tentative conclusion that dependence will have to be placed largely, if not exclusively, on levees for flood protection.

The construction of the Roosevelt Dam on the Salt River, in Arizona, has greatly improved flood conditions down stream, and the completion by the U. S. Bureau of Reclamation of the American Falls Reservoir on Snake River, Idaho, with a storage capacity of 1 700 000 acre-ft., has put an end to floods lower down.

An outstanding example of the possibilities of combining flood control and irrigation in streams of the arid region is the proposed Boulder Canyon Reservoir on the Colorado River. Here, the investigations of the Bureau of Reclamation engineers have demonstrated that by the construction of a dam in a narrow canyon raising the water surface 550 ft., a reservoir with a capacity of 26 000 000 acre-ft. may be formed, which would permit the utilization of the entire flow of the stream for irrigation and, in addition, would provide ample reserve capacity to regulate its floods to any desired extent. This would furnish flood protection for a number of irrigated valleys in the river bottoms between Needles, Calif., and Yuma, Ariz., and definitely remove the ever-present threat of disaster now hanging over Imperial Valley. This river is kept out of this basin by means of levees along its north bank, which turn it south into the Pacific Ocean, but water is carried from it by a canal which runs through Mexico into the Valley, where 400 000 acres are now irrigated and 700 000 acres can be irrigated. Sixty thousand people live on the irrigated lands of this Valley, watered from the Colorado, with all their homes

below sea level and from 100 to 200 ft. below the level of the river where it crosses the International Boundary.

The Valley has once been threatened by destruction through inundation. For a year the whole volume of the Colorado poured into this basin, flooding farms, washing away houses, and doing millions of dollars of damage. With the growing use of water along tributary streams, and the extension of its use in Mexico, further extension of irrigation is stopped, and the farms of the Valley are menaced by irreparable loss through drought. In September, 1924, less than one-third of the water needed by irrigators came down the river. There had been dangerous floods a few months before, followed by this devastating drought, that in two weeks caused a loss of $6 000 000 to the farmers.

The average flow of the Colorado River for a whole calendar year is about 16 000 000 acre-ft. This reservoir will hold, therefore, the entire discharge of the river for a year and a half. The great floods which now come down in the spring when the snows are melting will be caught here and held back, to be released later when water is needed to irrigate parched fields. No water will flow over the dam; all that goes down the stream will be discharged through its regulating gates which will open into tunnels that pass around the end of the dam.

POWER AS AFFECTING FLOOD CONTROL

F. W. SCHEIDENHELM,* M. AM. SOC. C. E.

Power development and flood control are so often incompatible that, from the standpoint of power development, one more readily states the subject of this paper in the inverted order, namely, flood control as affecting power development.

As a practical matter, consideration of power development in connection with flood control presupposes the use of reservoirs for such purposes. Some proponents of flood control conceive of benefits to power development as one of the economic justifications for control of floods by means of reservoirs. Proponents of water power development, on the other hand, are disposed to have misgivings whenever it is proposed to couple the use of storage capacity for flood control with that for power development. However, the two uses are not intrinsically, even if frequently, antagonistic, and it behooves one to analyze more deeply in order properly to appraise the effect of the one on the other.

For the purposes of this paper, the term, "floods", is assumed to include only those high waters which necessitate control in order to prevent damage. Further, the writer assumes that the control of Mississippi River floods solely by means of reservoirs is impracticable. This subject has been discussed in some detail in certain other papers of this Symposium.

To the extent that reservoir sites may be available below Cairo, Ill., such reservoirs are not likely to be of great value for power development. The reservoirs could be filled safely only if and when important floods required the temporary storage of flood-crest water and as soon as practicable thereafter would have to be emptied so as to be ready to receive the water of the next flood crest, which might occur during the very same season or might not occur for several years.

The heads, where sufficiently great, are likely to be so variable and the water supply so intermittent that the possibilities for power installations in connection with Lower Mississippi flood-control reservoirs in general do not seem attractive. Certainly one could not reasonably expect power developments to contribute toward the cost of such reservoirs.

In connection with the use of levees, spillways, or by-passes for Mississippi flood control, power development is not a consideration. Here, too, heads would not be sufficient, nor is any head available for a time sufficiently long.

This paper, therefore, must be concerned mainly with reservoirs located on the tributaries of the Mississippi River and particularly in the upper portions of the drainage areas of those tributaries, for it is primarily at the headwaters that the fall occurs and power must be developed. Unfortunately, it is in those very areas that storage reservoirs are least effective as regards the control of floods on the Mississippi proper.

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The relationship existing between power development and flood control in respect to such reservoirs will be considered in the following categories:

I.—Complete Regulation by Storage.
II.—Partial Regulation by Storage.
(1) Reservoirs Solely for Power Development.
(2) Reservoirs Solely for Flood Control.
(3) Reservoirs for Joint Power Development and Flood Control.

I.—Complete Regulation by Storage

At the extreme of favorable relationship between power and flood control is the situation existing when a stream is under complete regulation or control by storage. By "complete regulation", from the standpoints of power development and flood control, is herein meant approximate equalization of run-off as of a given point on a given stream, for instance, to the extent of causing approximately the same aggregate run-off to occur month after month, but with variation in flow, for power purposes, within the months. Such regulation may be the work of Man or of Nature. However, on any stream with as much as several thousand square miles of drainage area, complete regulation by artificial storage is not likely to be attempted for power purposes alone.

Whether the regulation be artificial or natural, and despite any utilization of the regulated run-off for power development, there can be practically no high stages of importance on such a stream save those due to ice gorges. This is true even if the stream flow be artificially regulated so as to yield little or no power at certain hours of the day, or days of a week, or weeks or seasons of a year, and correspondingly greater power during other hours, days, weeks, or seasons. In general, it is economically impracticable to install machinery of a capacity sufficient to utilize power from stream flow at such a rate as to produce a damaging flood wave.

The outstanding example of practically complete regulation is, of course, the St. Lawrence River, with its flow regulated by the Great Lakes. If one eliminates from consideration the "high water" caused by ice gorges, the St. Lawrence simply has no flood problem. The status would not be materially different were there artificial regulation for power purposes located, say, at the head of Niagara River, and were such regulation to release at times materially more than the normal flow.

The effect of the complete regulation of a tributary on a larger stream into which it discharges, constitutes a separate story. Generally, as regards flood control, that effect is favorable. Thus, the storing of water, especially of local flood run-off, cannot be otherwise than beneficial. Again, stored water on completely regulated head-waters of tributaries of the Mississippi would normally be released to offset the deficiencies in natural flow occurring either in mid-winter (for example, in the North), or in late summer or autumn (say, in the West, or in western regions of milder climate). In neither event can the water thus released from storage aggravate any major floods of the Mississippi, for apparently without exception these occur in the spring.

Whether complete regulation is feasible, whether it is economically warranted, and, if warranted, what proportions of cost should be borne by the respective beneficiaries thereof, are questions to be determined for each case, in the light of the facts of that case.

II.—Partial Regulation by Storage

Since the very question of the extent to which flood control is affected by power development implies some regulation by reservoirs, the category of partial regulation includes all pertinent cases other than those involving complete regulation.

(1) *Reservoirs Solely for Power Purposes.*—From the short-range standpoint, that is, as concerns areas down stream from the reservoirs, but not too far (say, within several hundred miles), such power reservoirs can only be beneficial in respect to flood control. This benefit, however, may be negligible and certainly is not to be relied on as adequate protection against floods.

If the storage capacity of a reservoir is utilized, then, axiomatically, the storage is drawn upon either when the natural stream flow is relatively low, or in advance and in anticipation of a flood. Conversely, the reservoir is usually refilled when the stream flow is above normal, preferably when the stream is in flood. The result can only be a tendency to reduce flood crests in near-by down-stream areas; certainly, no harm can result.

Such beneficial effect is increased if the storage capacity is sufficient to absorb the entire flood and if, at the same time, other power capacity is available for supplying market demands, so that during the flood the power use at the reservoir may be temporarily suspended in whole or in part. Indeed, if such other power capacity is "hydro" and is located at power stations on streams simultaneously in flood, the benefit from the standpoint of flood control would be accompanied by benefit from the standpoint of power development as well.

Even if the storage capacity of a reservoir be not utilized by drawing upon it, there is generally an absorption of flood peaks. Such absorption is not necessarily important in quantity, but is bound to be beneficial in quality. The absorption takes place without exception wherever the dam or spillway is not controlled by gates or siphons. It results from the fact that, as the flood discharge increases, the depth of flow over the spillway crest increases. This, in turn, necessitates the storing of water over the reservoir surface area to a depth corresponding to the increase in depth of over-flow. The water thus stored must necessarily include the run-off at the very peak of the flood. The result is to decrease the peak of the flood flow from the dam as compared with the peak of the flow into the reservoir.

The smaller the drainage area above the dam and the larger the surface of the reservoir, the greater is such flood-peak absorption. As an extreme there may be cited a drainage area of about 58 sq. miles proposed to be controlled by a storage dam for a power project in West Virginia. There

the reservoir area of about 6 450 acres will be relatively so great that, even assuming the reservoir to be filled at the beginning of the probable maximum flood, the actual flood peak passing over a spillway only 100 ft. long, would be hardly 5% of the natural peak at that point. A case applying to a larger drainage area, namely, 1 044 sq. miles, is that cited* by E. H. Sargent, M. Am. Soc. C. E., for the Sacandaga Reservoir with an area of about 26 000 acres. A flood of the same unit intensity and graph as the Miami River flood of 1913 apparently would have its peak reduced by one-half in passing through the Sacandaga Reservoir and over a spillway assumed to be 600 ft. long. In this case, too, the reservoir was assumed to have been full at the beginning of the flood.

The flood-peak water thus absorbed is retained only temporarily and upon subsidence of the flood is released gradually, but with the result of prolonging the period of high water.

From the long-range standpoint, that is, as concerns 1 000, or more, miles down stream from the reservoir, the effect may be beneficial, or it may be quite the contrary. If one considers the Mississippi River, and the fact that its flood peaks are due to varying coincidences of flood peaks from its several principal tributaries, it is clear that the retardation of flood-peak run-off from a given tributary might prevent the arrival of that peak run-off at a critical junction point, such as Cairo, at a time when it would aggravate a situation arising from synchronous arrivals of floods from other tributaries.

Conversely, the tremendous drainage area of the Mississippi and the long distances of flood travel on its tributaries make equally possible the circumstance that if a given head-water flood peak had not been disturbed or retarded, its water would have reached the critical junction head of the main coincident flood flows, whereas the retardation in fact aggravates the synchronism and hence the resultant Mississippi flood peak.

As regards the Mississippi, therefore, it seems fair to state that over a long period of time the beneficial and the harmful effects of local flood-peak absorption would offset each other. Ample evidence is available to show that the magnitude of effect in either direction can only be negligible.

Finally, it is well to bear in mind that an operating procedure which would involve drawing down a power reservoir in advance of the flood season, so as to afford some additional flood control, is likely to be detrimental to power development. It would involve reduction in head and, hence, in power and, should the anticipated flood not occur, there might be no opportunity to refill the depleted reservoir.

(2) *Reservoirs Solely for Flood Control.*—As to method of operation or purpose these may be classified, as follows:

(*a*) Reservoirs intended to prevent completely any flow past the dam until the reservoirs overflow.

The object is to postpone, completely if possible, the discharge of flood waters until after the flood flows from drainage areas farther down stream have passed the region to be protected. Such a reservoir may be intended

* *Transactions*, Am. Soc. C. E., Vol. 90 (June, 1927), p. 973.

to protect areas at considerable distances down stream, for instance, for the protection of Pittsburgh, at the junction of the Monongahela and Allegheny Rivers to form the Ohio River, in Western Pennsylvania, there has been proposed a system of reservoirs which, among others, would include reservoirs in West Virginia on the head-waters of the Monongahela River.

(*b*) Reservoirs or basins intended to detain the flood discharge, or, more accurately, to lessen the rate of flood discharge to such as may be carried safely by the stream channels through the areas to be protected.

Detention reservoirs are intended to protect areas relatively close to the reservoirs. The outstanding example is the system of the Miami Conservancy District, intended to protect the Cities of Dayton and Hamilton, Ohio, and certain smaller towns.

A reservoir of either type intended solely for flood control in general cannot reliably fulfill its function unless it be emptied as soon as practicable after a flood and thus made ready for the next flood. (Detention reservoirs cause this to take place automatically.) The result is that the flood flows completely withheld, or merely retarded, as the case may be, are released long before the natural stream flow reaches a stage materially below normal. In other words, the flood flows can not, must not, be utilized to increase natural flows during low-water periods. Consequently, benefits to power developments are necessarily incidental and, generally, are minor in amount.

It is conceivable that in regions where spring run-off and all floods are in large part due to the melting of snow, as in the Rocky Mountains, a flood-control reservoir might safely store for power or other purposes the last installments of spring run-off, once the melting of snow has been completed or has reached a predetermined stage.

(3) *Reservoirs for Joint Power Development and Flood Control.*—There have been various proposals for the construction of such reservoirs, some of these proposals having been made under the auspices of the Federal Power Commission. However, the writer knows of no actual instance of construction.

A clause for this purpose, proposed by the Federal Power Commission to be embodied in a license for a series of water power developments on a tributary of the Ohio River, reads as follows:

"If at any time any agency shall contribute funds to provide additional reservoir capacity for the purpose of storage of water for flood control, the licensee shall co-operate with such agency by constructing or permitting to be constructed, to the extent of the funds so contributed, additional reservoirs or additions to existing reservoirs as may be agreed between the licensee and such agency; and the licensee shall so operate its project works as to hold, or permit to be held, in reserve for flood control the storage capacity provided by funds so contributed."

The prospective licensee indicated that the clause would be satisfactory.

Usually, it is contemplated that the flood-control reservoir capacity be literally superimposed upon that intended for power uses. The object is to take advantage of favorable reservoir sites, especially in regions where favorable sites are scarce. Presumably, there are situations where the combination of the two objects can be attained at less expenditure for reservoir capacity

than would result from the construction of separate reservoirs for power and for flood control.

In effect, the combined reservoirs must be considered and operated as if they were separate. The danger is that the desires, or the needs, for power will tend to hold water too long in that part of the reservoir allocated to flood control. However, the penalty of mal-operation of the flood-control component of the reservoir is so great that power development ought not to be benefited more than it would be if the storage capacity allocated to flood control were actually in a separate reservoir.

Such a combination in a single reservoir would have the incidental effect of making impossible the development of power from the head which is represented by the depth of the flood-control component of the reservoir.

This subject is too involved for detailed treatment within the limits of this paper. However, in the light of the foregoing the following conclusions are offered:

1.—As regards the flood control of a given stream, power developments involving reservoirs on that stream generally have a beneficial effect and in especially favorable cases may have important beneficial effect.

2.—On streams which are under complete regulation by storage, that is, where approximate equalization of flow is attained, the requirements of both power development and of flood control on that stream may be met adequately and without conflict. Complete regulation is a desideratum, even if it is not a necessity, from both standpoints. (However, in the case of the Mississippi River it seems certain that adequate flood control, let alone complete regulation, cannot be obtained solely by means of storage reservoirs. Even for streams with drainage areas as small as several thousand square miles, complete regulation artificially must remain a rarity.)

3.—In the cases of streams which are under only partial regulation by storage, power developments may have favorable or unfavorable effects from the standpoint of flood control; in the main, however, such effects of power development will tend to be beneficial.

4.—So far as flood control by means of reservoirs involving only partial regulation of a given stream is concerned, the benefits which power developments even on that particular stream would derive are, in general, so limited in amount and so uncertain in occurrence that power could not economically contribute any substantial financial aid toward flood control.

5.—As regards the flood control of the Mississippi River, power is likely to be a minor consideration, both as to effects of power reservoirs in reducing Mississippi River floods and as to financial aid toward flood control.

THE EFFECT OF AGRICULTURAL DRAINAGE UPON FLOOD RUN-OFF

By Sherman M. Woodward* and Floyd A. Nagler,† Members, Am. Soc. C. E.

Synopsis

A survey of completed drainage enterprises in the Upper Mississippi Valley indicated that a comparison of stream-flow conditions prior and subsequent to extensive drainage could best be made upon the Des Moines and Iowa Rivers, a large portion of the water-sheds of which have been covered with artificial drains subsequent to the establishment of stream-gauging stations.

A critical examination of the records of these two streams shows that during flood periods there has been no significant change in their behavior which may be attributed to drainage. The total run-off from storms of like precipitation, the maximum rates of discharge, and the rain-water storage conditions within the basins seem to have been unaltered by the extensive drainage operations. It is believed that if any of these factors had been changed by a measurable amount, such fact could easily have been detected by the analysis made in this paper.

The drainage operations on the water-sheds of these two rivers involved the construction of tile drains, open ditches, and some straightening of stream channels, typical operations in drainage projects in the Upper Mississippi Valley. Since there has been no appreciable change in the run-off from these water-sheds which have experienced extensive drainage, it appears, therefore, that agricultural drainage has had a negligible effect upon the magnitude of either the total flow or the maximum discharge of the floods of the Mississippi River, less than 5% of the water-shed of which has witnessed agricultural drainage operations.

Although many references have been made in recent years to the effect of agricultural drainage upon flood run-off in general, and numerous statements have been published regarding the relation of such drainage to the magnitude and frequency of floods in the Mississippi and other rivers, almost no actual numerical evidence has hitherto been brought forward tending definitely to prove or disprove any of the relationships alleged.

This paper is a painstaking effort to determine whether existing records indicate the presence of any relationship or no relationship between agricultural drainage and flood run-off. All the data contained herein are based upon official Government records; the data concerning drainage are abstracted from the Fourteenth Census of the United States for 1920, Volume VII, Irrigation and Drainage; the data concerning run-off are taken from the *Water Supply Papers* of the U. S. Geological Survey; and the data on rainfall from the publications of the U. S. Weather Bureau.

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TABLE 30.—DRAINAGE STATISTICS FROM 1920 CENSUS REPORT.*

(*A*) DRAINAGE STATISTICS FOR UNITED STATES AND IOWA.

	In United States, in acres.	In State of Iowa, in acres.	Page reference in U. S. Census Report, 1920.
In operating enterprises	65 495 038	5 224 478	373
In non-operating enterprises	3 924 821	158 534	373
Completed in 1920	56 763 751	4 685 080	373
By open ditches only	43 658 485	1 294 243	376-377
By open ditches and levees	5 656 000	96 228	376-377
By only tile drains	4 951 152	1 415 176	376-377
By open ditches and tile drains	10 731 279	2 444 906	376-377
By ditches, tile, and levees	475 013	37 925	376-377

(*B*) DRAINAGE STATISTICS FOR MISSISSIPPI RIVER BASIN.

	AREA, IN ACRES.			Page reference in U. S. Census Report, 1920.
	In operating enterprises.	In non-operating enterprises.	Total.	
Upper Mississippi River	10 850 408	521 759	11 372 167	367
Missouri River	2 510 150	352 894	2 863 044	367
Ohio River	10 998 378	158 932	11 157 310	367
Lower Mississippi River	8 072 564	1 194 600	9 267 164	367
Total	32 431 500	2 228 185	34 659 685	.....

(*C*) DRAINAGE STATISTICS FOR DES MOINES and IOWA RIVER BASINS.

	DES MOINES RIVER BASIN.		IOWA RIVER BASIN.		Page reference in U. S. Census Report, 1920.
	Acres.	Square miles.	Acres.	Square miles.	
In operating enterprises	2 808 170	4 388	936 841	1 461	481 541
In non-operating enterprises	96 987	152	0	0	481
Total	2 905 157	4 540	936 841	1 461	.....

*Vol. VII, Irrigation and Drainage.

In the Census report a distinction is made between farm drainage on the one hand, that is, the small-scale drainage, such as tile drainage, for example, that can be carried out by an individual farmer independent of any action by his neighbors, and drainage enterprises on the other hand, those larger co-operative undertakings requiring the joint effort of more than one land owner to consummate the community enterprise. As all familiar with agricultural drainage on an extensive scale will know, this latter class of drainage work is much the more extensive of the two and is the class that may be expected to have the chief effect upon stream run-off. All the subsequent references to drainage contained herein pertain solely to this second class. Table 30 presents a summary of statistics on drainage operations within the United

States and those portions of the Mississippi River water-shed that were intensively studied in this investigation.

Fig. 29 is a diagram showing the rate of increase in the area embraced in drainage enterprises in the whole United States during the last fifty years. It will be noted that the rate of progress was relatively slow until 1900, and that its greatest rate of increase was between 1905 and 1915.

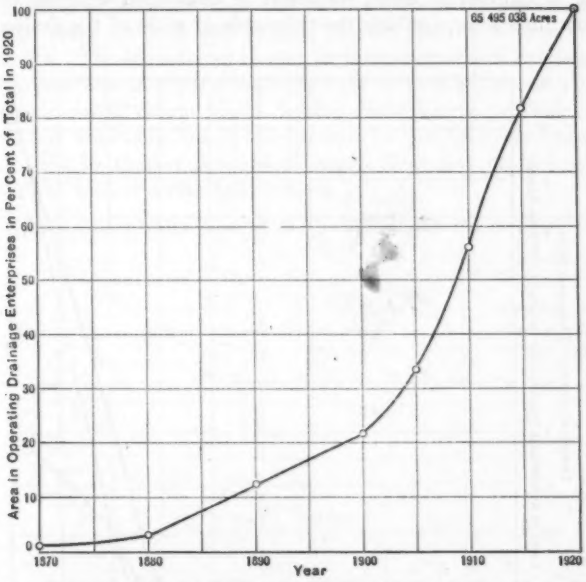

FIG. 29.—PROGRESS OF DRAINAGE IN THE UNITED STATES. FROM CENSUS OF 1920.

A map of the United States published in the 1920 Census report,* shows all the area included on January 1, 1920, in operating drainage enterprises, that is, enterprises the construction work of which had either been completed or begun. Of the total area included in these enterprises, about one-half is embraced in the Mississippi Valley. According to the Census figures between 4 and 5% of the total Mississippi River drainage area is included in operating drainage enterprises. Most of this is in the northern part of the country, that is, in the areas tributary to the Missouri, the Upper Mississippi, and the Ohio Rivers. Two large, compact, drained areas stand out conspicuously, the more eastern one in Ohio, Indiana, and Illinois, the other in North Central Iowa.

A study of the records shows that in the more easterly area much of the drainage work antedated extensive reliable stream measurements, thus precluding a satisfactory comparison of stream flow before and after drainage. A more satisfactory condition exists with reference to the Iowa area. In this region, drainage work was not extensive until after the passage of the 1904

* U. S. Census Rept., 1920, Pt. VII, p. 347.

Drainage Law, but nearly all the work was completed by 1917. The greater part of this compact area of drained Iowa land lies in the upper portions of the drainage areas of the Des Moines and the Iowa Rivers. Most fortunately for the purpose of this paper, stream measurements were begun upon these rivers by the U. S. Geological Survey in 1903. The progress of drainage in Iowa is shown in Fig. 30. For successive dates, the full line curve shows the area, in millions of acres, contained in authorized drainage enterprises; the broken lines show similarly the thousands of miles of tile drains and open ditches.

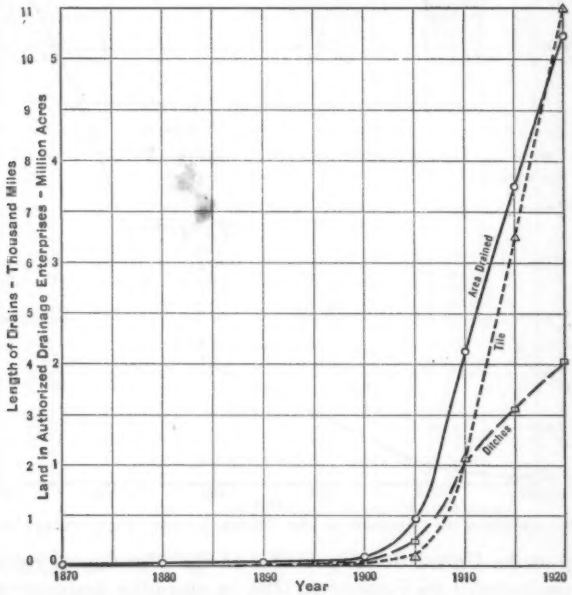

FIG. 30.—PROGRESS OF DRAINAGE IN IOWA. FROM U. S. CENSUS OF 1920.

After a careful scrutiny of the situation it was decided to be best to study the stream-flow records for a four-year period, from January 1, 1903, to December 31, 1906, to ascertain the behavior of these rivers prior to drainage, and to use the records similarly for the six-year period, January 1, 1918, to December 31, 1923, to learn their behavior subsequent to extensive drainage operations. Apparently, nowhere else in the United States are conditions so favorable for making this kind of comparison.

On the Iowa River a record of discharge has been maintained since 1903 at Iowa City, Iowa, above which point the drainage area is 3 140 sq. miles. Of this area, more than one-third is in operating drainage enterprises.

On the Des Moines River three gauging stations were started before the period of greatest drainage activity, and have been maintained to date (1927) with some fragmentary intermissions. The uppermost station was established

near Fort Dodge, Iowa, with a drainage area of 4 170 sq. miles, nearly all of which has been embraced in drainage enterprises. The second gauging station is at Des Moines, Iowa, with a tributary area of 6 180 sq. miles, about two-thirds of which has been drained. The lowest station is at Keosauqua, Iowa, embracing 13 900 sq. miles, more than one-third of which has been artificially drained.

Fig. 31 shows on an outline map of Iowa the position of the drainage areas of the Iowa and Des Moines Rivers, and the locations of the gauging stations. The drainage areas are long and narrow, that of the Des Moines extending into Minnesota. The shaded area is that portion embraced in drainage enterprises. The map also shows the isohyetal lines or rainfall contours for what is subsequently called Storm No. 1, lasting from May 8 to June 26, 1903. This storm was unusually heavy, the greatest in the four-year pre-drainage period, and will be further referred to later. The dots on the map show rainfall stations used in subsequent studies.

FIG. 31.—DRAINAGE AREAS OF DES MOINES AND IOWA RIVERS, SHOWING AREA OF OPERATING DRAINAGE ENTERPRISES IN 1920.

The northern part of these drainage areas is a flat prairie, with scanty, natural, shallow, drainage channels. Before artificial drainage a large portion of the land was in marshes or too wet in the spring for profitable cultivation.

The lower part of the drainage areas is rolling, with the highest portions of the land nearly 100 ft. above the rivers. Originally, most of this lower part was covered with hard-wood timber, but the greater portion of this forest has long been removed. The drainage improvements have included the construction of open ditches and tile drains as well as the deepening and straightening of natural channels.

Iowa streams yield in run-off less than one-quarter of the water which they receive in the form of precipitation. About three-quarters of this annual run-off is discharged during periods of high water, and it is the high-water periods only that make any significant contributions to Mississippi River floods. If the carrying out of agricultural drainage affects the flow of streams in any measurable way, such effects must be most conspicuous near the drained areas. More remote points down stream would naturally be less affected, in spite of any cumulative result from many tributaries. If no effect can be perceived in the streams traversing the drained areas, it may reasonably be concluded that no similar effects are produced at remote points on the larger rivers.

As a first preliminary study, the monthly run-off was compared with the monthly precipitation during the spring months of March, April, and May, and all other single months in which the stream flow was exceptionally large during the adopted pre-drainage and post-drainage periods. In this study, the precipitation was computed by taking the arithmetical average of that recorded at selected rainfall stations distributed over each basin so that each station represented an area of about 800 sq. miles. On the Iowa River five stations were used; on the Des Moines, five were used above Fort Dodge, seven above Des Moines, and fourteen above Keosauqua. These values of monthly precipitation and run-off are shown in Table 31 and in Fig. 32, in order that the stream behavior prior to drainage may be compared with that subsequent thereto.

The quantity of water appearing as run-off from a given amount of precipitation is no doubt larger during the early spring months than later, decreasing with the increase in temperature and the growth of vegetation. This fact was taken into consideration by plotting the monthly values for each gauging station in three separate groups. One group includes the records for March and a few for February; the second group includes those for the month of April; and the third group includes those for the month of May, and a few for the months of June, July, and September. In this, as in subsequent diagrams, the values of run-off prior to drainage, have been indicated by solid symbols, in contrast to the open symbols used for the post-drainage period.

The plotting of data such as these naturally results in a wide scattering of points, because the run-off for a given month may have been the result of precipitation which fell during the previous month, to say nothing of the various hydrologic factors which may have caused variations in the stream yield. The points showing the run-off previous to drainage seem to group themselves well within and about the system of points obtained for the post-drainage period, and no other conclusion is possible than that the average

monthly yield of the streams subsequent to drainage does not seem to be different from that previous to drainage. If any difference can be detected from this plotting, it would appear to indicate that the streams yielded a greater quantity during the flood months of 1903 to 1906 than during those of 1918 to 1923. Certainly, other factors than drainage have been more significant in producing differences in stream yield during both the pre-drainage and the post-drainage period.

TABLE 31.—MONTHLY PRECIPITATION AND RUN-OFF ON DES MOINES AND IOWA RIVERS.

Year.	Month.	Des Moines River.						Iowa River.	
		Above Fort Dodge.		Above Des Moines.		Above Keosauqua.		Above Iowa City.	
		Precipitation, in inches.	Run-off, in inches.	Precipitation, in inches.	Run-off, in inches.	Precipitation, in inches.	Run-off, in inches.	Precipitation, in inches.	Run-off, in inches.
BEFORE DRAINAGE.									
1903	March	.....	.....	0.95	1.02	.....	.....	.....	.....
	April	.....	.....	2.69	0.67	.....	.....	.....	.....
	May	.....	.....	9.96	1.88	9.65	.....	7.39	.....
	June	.....	.....	3.29	1.81	3.63	2.69	2.45	2.59
1904	March	.....	.....	.....	.....	.....	.....	.....	.....
	April	.....	.....	.....	.....	4.37	1.25	3.54	0.82
	May	.....	.....	.....	.....	3.22	0.61	3.69	0.58
1905	March	.....	.....	.....	.....	2.12	1.03	2.12	1.35
	April	.....	.....	.....	.....	3.42	0.66	2.91	0.70
	May	7.37	0.99	.....	.....	5.76	1.04	7.12	1.73
	June	4.58	0.71	5.08	0.75	5.66	1.00	4.59	1.03
1906	February	.....	.....	.....	.....	1.23	0.59	.....	.....
	March	.....	.....	2.33	0.89	2.12	0.86	2.60	1.02
	April	1.65	1.08	2.08	1.09	2.40	1.14	2.29	1.69
	May	4.42	0.64	3.92	0.64	3.18	0.52	3.08	0.64
AFTER DRAINAGE.									
1918	March	.....	.....	.....	.....	.....	.....	.....	.....
	April	1.54	0.15	1.58	0.15	1.76	0.10	2.70	0.23
	May	7.43	0.28	6.92	0.27	6.41	0.31	8.32	1.30
	June	4.53	0.82	5.34	1.75	5.08	1.35	7.53	3.90
1919	March	.....	.....	1.83	0.86	2.20	1.04	2.37	1.46
	April	4.78	1.68	5.15	1.81	5.14	1.45	5.14	1.70
	May	2.17	1.12	2.21	1.21	2.62	1.35	3.07	1.88
	June	6.47	1.68	6.67	1.60	7.05	1.67	6.43	1.74
	July	3.62	0.80	3.28	0.68	3.41	0.59	1.68	0.47
1920	March	2.21	1.13	2.37	1.03	2.52	1.28	3.30	1.52
	April	3.81	0.77	3.84	0.84	4.31	1.24	4.31	1.50
	May	2.29	1.05	2.37	1.08	2.67	1.27	3.65	1.54
	July	5.92	1.83	6.01	1.49	5.35	1.08	4.03	0.55
1921	February	0.97	0.42	0.99	0.49	0.96	0.39	0.89	0.42
	March	1.30	0.40	1.24	0.38	1.36	0.31	1.49	0.42
	April	2.01	0.39	2.20	0.44	2.84	0.57	4.11	0.63
	May	7.17	0.77	6.03	0.71	4.89	0.61	4.31	0.73
	September	5.74	0.37	6.03	0.66	6.50	0.87	7.79	0.53
1922	March	1.17	0.60	1.40	0.52	2.01	0.63	1.91	0.98
	April	3.74	0.90	3.53	0.95	3.20	1.00	3.09	1.20
	May	0.88	0.31	2.90	0.42	3.57	0.57	4.11	0.58
1923	March	1.56	0.30	2.90	0.39	2.54	0.62	3.25	0.54
	April	2.29	0.24	2.14	0.38	1.88	0.48	1.60	1.26
	May	2.59	0.12	3.06	0.21	3.01	0.31	2.48	0.28

In order to make a more delicate and conclusive comparison of the behavior of the streams before and after drainage than that presented in Fig. 32, every flood of significance which occurred on these streams during these periods, was next studied in considerable detail. Each flood that produced a maximum run-off greater than 2 sec-ft. per sq. mile was included in this investigation. The total run-off was computed from the beginning of the flood until the dis-

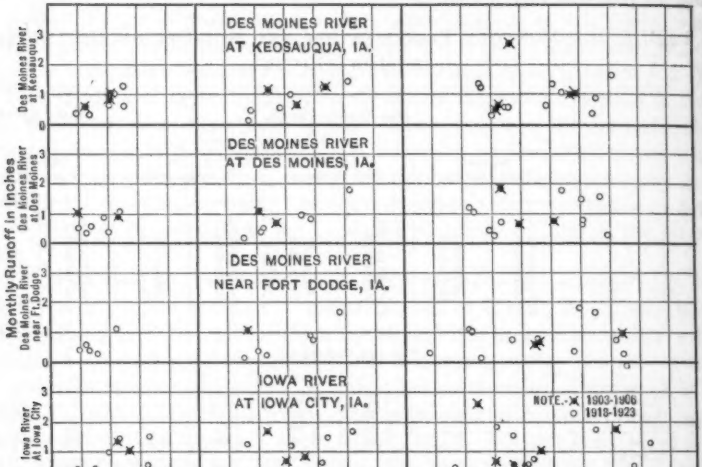

FIG. 32.—RELATION OF PRECIPITATION TO RUN-OFF IN MONTHS OF HIGH WATER ON THE DES MOINES AND IOWA RIVERS, COMPARING 1903-1906 WITH 1918-1923.

charge had diminished to a flow of ½ sec-ft. per sq. mile, or lower, and the average precipitation producing this high water was evaluated from all rainfall stations within and about the water-sheds in question. Nine such floods occurred during the pre-drainage period, 1903 to 1906, and eleven during the post-drainage years, 1918 to 1923. Individual hydrographs of these floods are shown in Figs. 33 and 34.

The average precipitation was computed by applying the method suggested by Thiessen* to obtain the true weighted average precipitation on the water-shed. All available precipitation stations were used, the number considered for each water-shed varying somewhat from year to year. The maximum and minimum numbers were as given in Table 32.

Isohyetal lines were also drawn, for each stormy period, upon maps of the drainage area in order to visualize the distribution of the storm. Other values pertinent to the study of these storms and their yield have been given in Tables 33 and 34.

In order to study the amount of run-off produced by a given amount of "effective precipitation" these quantities for each storm period were plotted

* *Journal*, New England Water Works Assoc., March, 1924, p. 26.

on Fig. 35, employing the same general scheme of symbols used previously in Fig. 32. In order to facilitate this study these storms were first classified into three groups according to the average temperature which existed during the storm period. One group, with average temperatures below 45°, includes all early spring storms coming before the growing season had started; a second group, with average temperatures between 45° and 60°, includes those storms

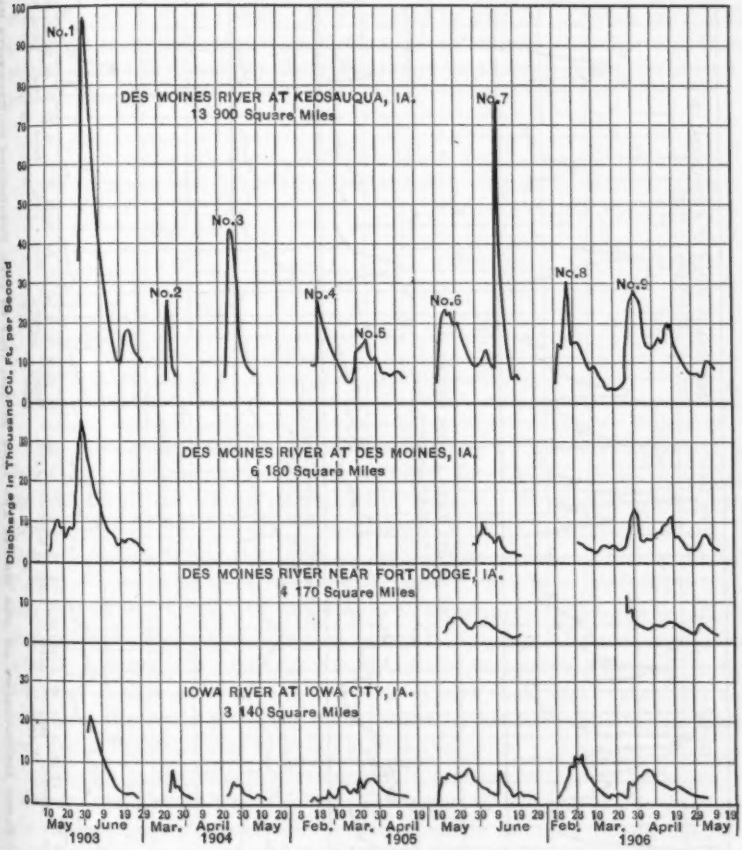

FIG. 33.—FLOOD HYDROGRAPHS ON DES MOINES AND IOWA RIVERS, 1903 TO 1906, PREVIOUS TO EXTENSIVE DRAINAGE.

which occurred during the early growing season; the third group, with average temperatures exceeding 60°, includes the early and late summer storms which occurred in June, July, and September, the storms which generally include the most intense rainfall and produce the highest floods on these streams. Some distinction was made in plotting the run-off from these storms, depending upon the location of the greatest amount of rainfall on the basin.

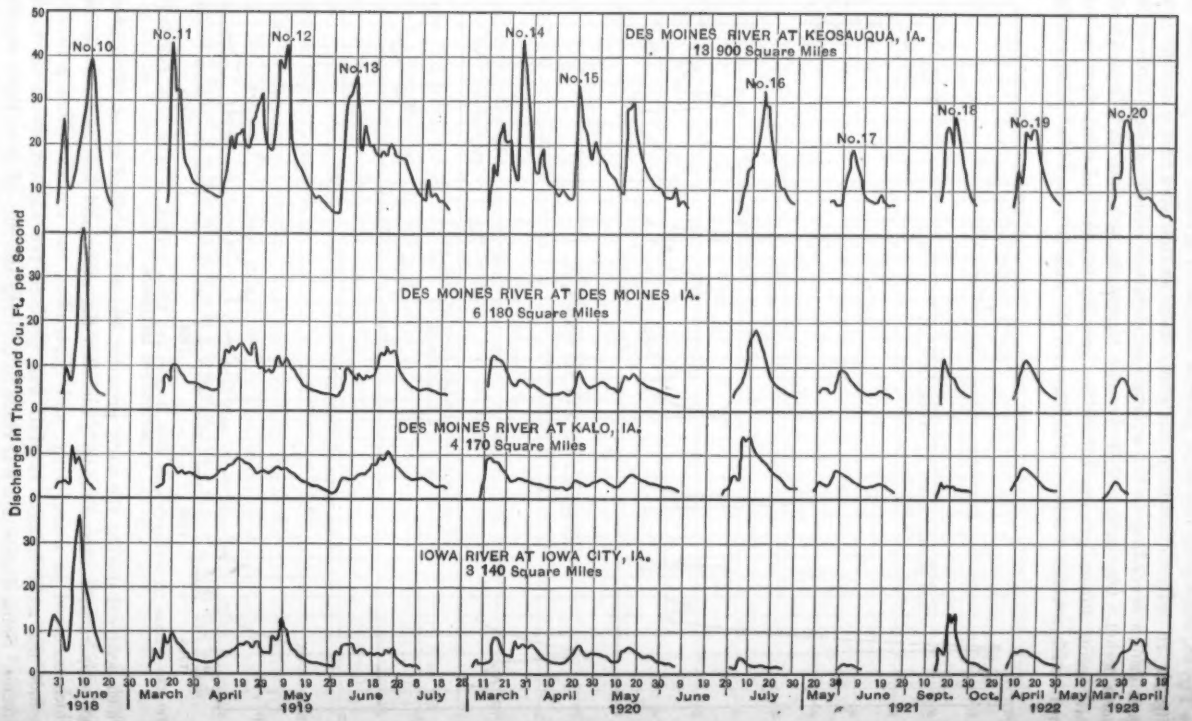

FIG. 84.—FLOOD HYDROGRAPHS ON DES MOINES AND IOWA RIVERS, 1918 TO 1923, SUBSEQUENT TO EXTENSIVE DRAINAGE.

Triangles are used to indicate the storms in which the heaviest rainfall occurred on the upper part of the drainage area; circles to indicate storms near the center, or spread uniformly over the whole drainage area; and squares to indicate storms heaviest on the lower portion of the basin. The lines showing the average relation between rainfall and run-off were drawn from the point of zero rainfall and precipitation through the area covered by the open circles, representing those storms which were of a general nature or concentrated about the center of the basin. It was noted in plotting the data on storms subsequent to drainage that those in the upper basin gave a lower run-off than general storms (indicated by the circles), and that storms in the lower basin gave a higher run-off.

TABLE 32.

Water-shed.	Number of Precipitation Stations.	
	Maximum.	Minimum.
Des Moines River:		
Above Fort Dodge	13	9
Above Des Moines	21	17
Above Keosauqua	45	30
Iowa River:		
Above Iowa City	16	12

The dark symbols on Fig. 35 indicate the run-off of the floods previous to drainage. In general, they fit in very well with the system of points indicating the behavior of the stream subsequent to drainage. If any difference exists there is some indication that storms concentrated on the upper portions of these water-sheds produced a larger run-off previous to drainage than at present (1927). The correlation of rainfall and run-off in Fig. 35 is much better than that secured in Fig. 32, and would seem to be adequate to detect any significant difference which may have been caused by drainage. It is apparent from Fig. 35, however, that drainage has caused no measurable difference in the total flow of these streams during periods of high water and hence could not have been effective in producing higher stages in the Lower Mississippi River, only 4% of the water-shed of which has been subjected to drainage operations.

Although no significant difference can be detected in the total flood run-off of these Iowa streams since drainage in contrast to that previous to drainage, the statement has often been made that certainly the peak run-off has been increased by these operations. It is difficult, indeed, to correlate the maximum flow of a storm with any single hydrologic factor, since it is dependent upon so many. Nevertheless, the maximum average 24-hour rate of discharge during these twenty periods of high water was plotted in Fig. 36 against the average daily precipitation during the storm which produced the run-off. Symbols similar to those in Fig. 35 were used, and lines were drawn from the point of zero rainfall and run-off through the open circles which represent

FIG. 34.—FLOOD HYDROGRAPHS ON DES MOINES AND IOWA RIVERS, 1918 TO 1923, SUBSEQUENT TO EXTENSIVE DRAINAGE.

TABLE 33.—SUMMARY OF FLOODS ON DES MOINES RIVER.

Flood No.	Date of Storm.			Duration of storm, in days.	Number of stormy periods.	Number of days of more than 0.25 in. of rainfall.	Total effective rainfall, all days, with excess of 0.25 in.	Total rainfall, in 30 days before storm.	Date of maximum rainfall.	Maximum 24-hour rainfall.	Location of storm center.	Average temperature at Des Moines during storm, in degrees Fahrenheit.	Time of Flood.		Duration of flood, in days.	Total run-off during flood, in inches.	Date of maximum flow.	Maximum 24-hour flow, in second-feet.	Ratio, run-off to rainfall.	Average rate of rainfall, in inches per day.	Ratio, maximum flow to average flow.
	Year.	Time. Beginning.	Time. End.										Beginning.	End.							
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)
Floods on Des Moines River Above Fort Dodge, Iowa—Period Before Drainage.																					
6	1905	May 9	June 7	30	4	6 to 7	6.81	1.66	May 10	2.20	*GU*	62	May 12	June 9	29	1.24	May 17–18	6 460	0.182	0.221	1.34
7	1905	June 8	June 21	14	2	2 to 4	1.63	6.71	June 18	1.77	*L*	71	June 10	June 21	12	0.20	June 8	3 450	0.123	0.136	1.81
9	1906	Mar. 22	May 4	44	5	1 to 8	2.68	0.96	May 1	2.00	*U*	50	Mar. 26	May 10	47	1.74	Mar. 26	12 000	0.650	0.061	2.85
Floods on Des Moines River Above Fort Dodge, Iowa—Period After Drainage.																					
10	1918	May 26	June 7	13	2	4 to 6	4.53	4.69	June 4	3.35	*MU*	70	May 28	June 13	17	0.78	June 4	11 400	0.172	0.348	2.20
11	1919	Mar. 9	Apr. 5	28	3	1 to 3	0.96	1.66	Mar. 15	1.33	*L*	45	Mar. 13	Apr. 6	25	1.16*	Mar. 17–20	7 330	1.21	0.094	1.41
12	1919	Apr. 6	May 31	56	6	6 to 12	5.27	1.60	Apr. 7	2.10	*GL*	55	Apr. 7	May 31	55	2.58	Apr. 21	7 970	0.490	0.094	1.50
13	1919	June 1	July 18	48	8	9 to 12	8.75	1.89	July 4	3.10	*MU*	75	June 1	July 21	51	2.35	June 24–25	10 200	0.268	0.182	1.96
14	1920	Mar. 10	Apr. 12	34	8	2 to 6	2.44	0.91	Mar. 11	1.45	*L*	43	Mar. 10	Apr. 17	39	1.50	Mar. 13–14	9 000	0.615	0.072	2.09
15	1920	Apr. 14	June 7	55	10	3 to 11	5.65	2.93	Apr. 19	1.68	*G*	58	Apr. 18	June 7	51	1.57	May 17	5 350	0.278	0.103	1.57
16	1920	June 27	July 27	31	5	7 to 10	6.30	2.17	July 6	2.30	*UL*	74	June 29	July 30	32	1.86	July 7–8	13 700	0.295	0.208	2.11
17	1921	May 16	June 22	38	5	5 to 11	6.28	2.49	May 26	2.79	*U*	75	May 19	June 22	35	1.18	May 28	6 600	0.188	0.165	1.73
18	1921	Sept. 12	Sept. 25	14	3	3 to 5	3.71	3.21	Sept. 16	3.36	*U*	69	Sept. 16	Sept. 27	12	0.28	Sept. 16	3 380	0.075	0.265	1.30
19	1922	Apr. 5	Apr. 25	21	3	2 to 3	2.44	1.88	Apr. 9	2.25	*GM*	51	Apr. 9	Apr. 27	19	0.75	Apr. 18	7 340	0.307	0.116	1.67
20	1923	Mar. 11	Mar. 22	12	1	0 to 3	0.61	0.93	Mar. 15	1.00	*GL*	27	Mar. 23	Mar. 29	7	0.19	Mar. 25	4 160	0.311	0.051	1.39

NOTE.—*U* = greatest precipitation above Fort Dodge; *M* = greatest precipitation near Des Moines; *L* = greatest precipitation below Des Moines; and *G* = general distribution of precipitation.

* Winter storage and many light rains resulted in run-off exceeding rainfall.

TABLE 33.—(*Continued.*)

Flood No.	Date of Storm. Year.	Time. Beginning.	Time. End.	Duration of storm, in days.	Number of stormy periods.	Number of days of more than 0.25 in. of rainfall.	Total effective rainfall, all days, with excess of 0.25 in.	Total rainfall, in 30 days before storm.	Date of maximum rainfall.	Maximum 24-hour rainfall.	Location of storm center.	Average temperature at Des Moines during storm, in degrees Fahrenheit.	Time of Flood. Beginning.	Time of Flood. End.	Duration of flood, in days.	Total run-off during flood, in inches.	Date of maximum flow.	Maximum 24-hour flow, in second-feet.	Ratio, run-off to rainfall.	Average rate of rainfall, in inches per day.	Ratio, maximum flow to average flow.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)
Floods on Des Moines River Above Des Moines, Iowa—Period Before Drainage.																					
1	1903	May 8	June 26	50	6	10 to 20	9.14	3.17	May 25	3.70	*GM*	65	May 12	June 30	50	3.45	May 30	36 000	0.378	0.188	3.13
6	1905	May 9	June 7	30	4	5 to 8	6.54	2.06	May 10	2.20	*GU*	62	May 27†	June 9	..	0.51†	June 2	8 800	0.078†	0.218	1.44†
7	1905	June 8	June 21	14	2	2 to 4	1.82	6.32	June 10	3.49	*L*	71	June 10	June 21	12	0.21	June 10	6 750	0.115	0.130	2.33
8	1906	Feb. 11	Mar. 19	37	4	1 to 4	1.18	0.31	Feb. 13	1.45	*GL*(?)	28	Mar. 1†	Mar. 23	..	0.52†	.....	.....	0.440†	0.032	....
9†	1906	Mar. 22	May 4	44	5	1 to 8	3.44	1.08	May 1	2.00	*U*	50	Mar. 24	May 10	48	1.85	Mar. 30	13 700	0.538	0.078	2.14
Floods on Des Moines River Above Des Moines, Iowa—Period After Drainage.																					
10	1918	May 26	June 7	13	2	4 to 9	5.57	4.63	June 4	3.85	*MU*	70	May 30	June 17	19	1.65	June 7	41 000	0.296	0.428	2.85
11	1919	Mar. 9	Apr. 5	28	3	1 to 3	1.21	1.79	Mar. 15	2.44	*L*	45	Mar. 15	Apr. 7	24	0.89	Mar. 19-20	10 000	0.735	0.043	1.61
12	1919	Apr. 6	May 31	56	6	6 to 12	5.64	1.80	Apr. 7	2.10	*GL*	55	Apr. 8	May 31	54	2.83	Apr. 24	15 300	0.502	0.101	1.76
13	1919	June 1	July 18	48	8	7 to 13	8.06	1.94	July 4	3.10	*MU*	75	June 1	July 20	48	2.13	June 23	14 000	0.264	0.168	1.89
14	1920	Mar. 10	Apr. 12	34	8	2 to 7	2.24	0.81	Mar. 11	1.46	*L*	43	Mar. 12	Apr. 18	38	1.36	Mar. 14-15	12 000	0.607	0.066	2.00
15	1920	Apr. 14	June 7	55	10	3 to 11	5.74	3.01	May 11	2.20	*G*	58	Apr. 19	June 7	50	1.64	May 17	8 260	0.286	0.104	1.50
16	1920	June 27	July 27	31	5	5 to 10	6.01	1.86	July 6	2.80	*UL*	74	July 2	July 29	28	1.43	July 12	18 200	0.238	0.194	2.14
17	1921	May 16	June 22	38	5	5 to 11	5.73	2.71	May 26	2.79	*U*	75	May 21	June 22	33	1.02	May 30	9 430	0.178	0.151	1.81
18	1921	Sep. 12	Sept. 25	14	3	3 to 5	4.10	4.30	Sept. 16	3.36	*U*	69	Sept. 16	Sept. 30	15	0.56	Sept. 17	11 400	0.137	0.293	1.84
19	1922	Apr. 5	Apr. 25	21	3	1 to 4	2.32	1.50	Apr. 9	2.72	*GM*	51	Apr. 10	Apr. 27	18	0.83	Apr. 13-14	11 500	0.358	0.110	1.51
20	1923	Mar. 11	Mar. 22	12	1	0 to	0.84	1.06	Mar. 15	1.00	*GL*	27	Mar. 24	Apr. 1	9	0.32	Mar. 28	8 000	0.381	0.070	1.33

† Incomplete record of flow.

TABLE 33.—(*Continued.*)

Flood No.	Date of Storm. Year.	Time. Beginning.	Time. End.	Duration of storm, in days.	Number of stormy periods.	Number of days of more than 0.25 in. of rainfall.	Total effective rainfall, all days, with excess of 0.25 in.	Total rainfall, in 30 days before storm.	Date of maximum rainfall.	Maximum 24-hour rainfall.	Location of storm center.	Average temperature at Des Moines during storm, in degrees Fahrenheit.	Time of Flood. Beginning.	Time of Flood. End.	Duration of flood, in days.	Total run-off during flood, in inches.	Date of maximum flow.	Maximum 24-hour flow, in second-feet.	Ratio, run-off to rainfall.	Average rate of rainfall, in inches per day.	Ratio, maximum flow to average flow.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)
Floods on Des Moines River Above Keosauqua, Iowa—Period Before Drainage.																					
1	1903	May 8	June 26	50	6	9 to 12	11.69	3.04	May 27	4.02	*G M*	65	May 29*	June 30	..	3.04*	June 1	97 140	0.26+*	0.234	2.82*
2	1904	Mar. 24	Mar. 25	2	1	1 to 2	0.54	0.90	Mar. 24	2.00	*G L*	36	Mar. 25	Mar. 28	4	0.16	Mar. 25	21 840	0.296	0.270	1.44
3	1904	Apr. 20	May 9	20	2	2 to 6	2.87	3.21	Apr. 25	3.09	*L*	57	Apr. 25	May 9	15	0.82	Apr. 26-27	43 300	0.286	0.144	2.11
4	1905	Feb. 23	Mar. 16	22	2	1 to 3	0.42	1.54	Feb. 24 Mar. 10-12	1.10	*U*	37	Feb. 25	Mar. 18	22	0.73	Mar. 1	26 230	1.74 †	0.019	2.11
5	1905	Mar. 17	Apr. 11	26	4	2 to 5	2.20	0.76	Apr. 2	2.06	*G*	51	Mar. 19	Apr. 13	26	0.69	Mar. 25	16 530	0.313	0.085	1.65
6	1905	May 9	June 7	30	4	6 to 7	5.47	2.92	May 10	2.50	*G U*	62	May 11	June 9	30	1.18	May 15	23 360	0.329	0.182	1.59
7	1905	June 8	June 21	14	2	2 to 4	2.59	5.75	June 10	11.09	*L*	71	June 10	June 21	12	0.66	June 10	75 750	0.255	0.185	3.80
8	1906	Feb. 11	Mar. 19	35	4	2 to 6	1.40	0.40	Feb. 18	1.48	*G L*	28	Feb. 21	Mar. 25	33	0.91	Feb. 25	30 800	0.650	0.038	2.99
9	1906	Mar. 22	May 4	44	5	4 to 8	3.51	1.30	May 1	2.00	*U*	50	Mar. 26	May 10	46	1.79	Mar. 30	28 000	0.510	0.080	1.93
Floods on Des Moines River Above Keosauqua, Iowa—Period After Drainage.																					
10	1918	May 26	June 7	13	2	4 to 7	5.37	4.55	May 27	3.43	*M U*	70	May 28	June 18	22	1.21	June 10-11	39 300	0.225	0.413	1.90
11	1919	Mar. 9	Apr. 5	28	3	1 to 3	1.91	1.87	Mar. 15	2.44	*L*	45	Mar. 15	Apr. 8	25	1.07	Mar. 17	43 700	0.560	0.068	2.73
12	1919	Apr. 6	May 31	56	6	8 to 13	6.21	2.57	Apr. 9	2.13	*G L*	55	Apr. 9	May 31	53	2.63	May 8	41 800	0.418	0.111	2.27
13	1919	June 1	July 18	48	8	7 to 13	8.38	2.44	June 10	5.10	*M U*	75	June 1	July 20	50	2.16	June 8	85 900	0.258	0.175	2.23
14	1920	Mar. 10	Apr. 12	34	8	3 to 8	2.77	0.66	Mar. 24	2.07	*L*	43	Mar. 12	Apr. 18	38	1.70	Mar. 26	44 100	0.614	0.081	2.64
15	1920	Apr. 14	June 7	55	10	8 to 11	6.46	3.09	Apr. 19	3.34	*G*	58	Apr. 19	June 7	50	2.13	Apr. 21	33 300	0.330	0.117	2.10
16	1920	June 27	July 27	31	3	6 to 9	5.60	2.02	July 14	3.32	*U L*	74	July 2	July 27	26	1.01	July 14	33 200	0.180	0.207	2.03
17	1921	May 16	June 22	38	5	0 to 3	4.81	3.30	May 26	2.79	*U*	75	May 24	June 22	30	0.76	June 3	18 900	0.158	0.126	1.99
18	1921	Sept. 12	Sept. 25	14	3	5 to 6	3.89	4.93	Sept. 16	3.26	*U*	69	Sept. 16	Sept. 30	15	0.67	Sept. 21	26 900	0.173	0.278	1.60
19	1922	Apr. 5	Apr. 25	21	3	2 to 6	2.42	1.92	Apr. 9	2.72	*G M*	51	Apr. 8	Apr. 28	21	0.84	Apr. 17	23 500	0.347	0.115	1.57
20	1923	Mar. 11	Apr. 16	37	4	2 to 6	1.90	1.15	Mar. 11	1.51	*G L*	36	Mar. 23	Apr. 19	28	0.88	Mar. 28	26 200	0.487	0.051	2.26

* Incomplete record of flow. † Winter storage and light rains resulted in a run-off exceeding effective rainfall.

TABLE 34.—SUMMARY OF FLOODS ON IOWA RIVER AT IOWA CITY, IOWA.

Flood No.	Date of Storm. Year.	Time. Beginning.	Time. End.	Duration of storm, in days.	Number of stormy periods.	Number of days of more than 0.25 in. of rainfall.	Total effective rainfall, all days excess of 0.25 in.	Total rainfall in 30 days before storm.	Date of maximum rainfall.	Maximum 24-hour rainfall.	Location of storm center.	Average temperature during storm at Des Moines, in degrees Fahrenheit.	Time of Flood. Beginning.	Time of Flood. End.	Duration of flood, in days.	Total run-off during flood, in inches.	Date of maximum flow.	Maximum 24-hour flow, in second-feet.	Ratio, run-off to rainfall.	Average rate of rainfall, in inches per day.	Ratio, maximum flow to average flow.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)
PERIOD BEFORE DRAINAGE.																					
1	1903	May 8	June 26	50	6	9 to 12	8.30	3.04	May 25	3.63	*G*	65	June 1*	June 28	....	2.62*	June 3	21 500	0.316*	0.166	3.67
2	1904	Mar. 24	Apr. 1	9	2	1 to 2	1.24	1.38	Mar. 24	1.70	*G*	36	Mar. 25	Apr. 4	11	0.42	Mar. 25	8 800	0.339	0.138	2.67
3	1904	Apr. 20	May 9	20	2	2 to 6	3.08	2.77	Apr. 25	3.00	*L*	57	Apr. 23	May 12	20	0.66	Apr. 26	5 510	0.214	0.154	1.97
4	1905	Feb. 23	Mar. 16	22	2	1 to 3	0.82	1.38	Feb. 26	1.00	*G*	37	Feb. 23	Mar. 18	24	0.64	Mar. 12	4 700	0.783	0.037	2.14
5	1905	Mar. 17	Apr. 11	26	4	2 to 5	1.81	1.23	Mar. 19	1.75	*GM*	51	Mar. 19	Apr. 13	26	1.14	Mar. 25	6 500	0.630	0.070	1.76
6	1905	May 9	June 7	30	4	6 to 7	5.71	2.69	May 11	3.48	*GL*	62	May 11	June 9	30	1.91	May 24	8 540	0.335	0.190	1.58
7	1905	June 8	June 21	14	2	2 to 4	2.91	6.16	June 10	4.35	*L*	71	June 10	June 27	18	0.66	June 10	8 110	0.227	0.208	2.90
8	1906	Feb. 11	Mar. 19	37	4	2 to 6	1.37	0.84	Feb. 13	1.43	*GL*	28	Feb. 19	Mar. 25	35	2.05	Mar. 2	11 400	1.50 †	0.037	2.33
9	1906	Mar. 22	May 4	44	5	4 to 8	3.80	1.32	Mar. 26	1.37	*U*	50	Mar. 26	May 5	41	2.21	Apr. 3–7	7 960	0.582	0.086	1.73
PERIOD AFTER DRAINAGE.																					
10	1918	May 26	June 7	13	2	4 to 7	6.57	6.90	June 4	3.73	*UM*	70	May 26	June 19	26	4.22	June 8	35 300	0.643	0.505	2.58
11	1919	Mar. 9	Apr. 5	28	3	1 to 3	2.03	2.01	Mar. 16	1.55	*GM*	45	Mar. 10	Apr. 9	31	1.61	Mar. 16	8 820	0.783	0.072	2.00
12	1919	Apr. 6	May 31	56	6	8 to 13	7.20	2.52	Apr. 7	2.01	*GU*	55	Apr. 10	May 31	52	3.26	May 7	12 800	0.453	0.129	2.41
13	1919	June 1	July 18	38	7	7 to 13	7.32	2.84	June 17	3.27	*M*	75	June 1	July 18	48	2.09	June 6–7	6 940	0.286	0.152	1.87
14	1920	Mar. 10	Apr. 12	34	8	3 to 8	3.25	0.65	Mar. 25	1.61	*G*	43	Mar. 6	Apr. 16	42	2.20	Mar. 17–18	8 130	0.677	0.096	1.85
15	1920	Apr. 14	June 7	55	10	8 to 11	6.31	3.04	May 12	2.29	*G*	58	Apr. 17	June 7	52	2.49	May 16	6 450	0.395	0.115	1.61
16	1920	June 27	July 27	31	5	6 to 9	4.86	2.11	July 5	2.63	*U*	74	July 2	July 27	26	0.57	July 8	3 550	0.117	0.157	1.97
17	1921	May 29	June 8	11	3	0 to 3	0.94	3.55	May 30	1.20	*G*	70	June 1	June 8	8	0.21	June 2	2 680	0.223	0.085	1.22
18	1921	Sept. 12	Sept. 25	14	3	5 to 6	5.47	5.27	Sept. 16	5.26	*M*	69	Sept. 16	Oct. 8	23	1.65	Sept. 23	14 800	0.302	0.390	2.35
19	1922	Apr. 5	Apr. 25	21	3	2 to 6	2.53	2.18	Apr. 9	1.36	*G*	51	Apr. 9	May 1	23	1.05	Apr. 11	5 780	0.415	0.120	1.48
20	1923	Mar. 11	Apr. 16	37	4	2 to 6	2.67	1.16	Mar. 15	1.27	*GL*	36	Mar. 27	Apr. 19	24	1.33	Apr. 7	8 420	0.498	0.072	1.79

NOTE.—*L* = near Iowa City; *M* = near Marshalltown; *U* = above Iowa Falls; and *G* = general. *Incomplete record of stream flow. † Winter storage and light rains resulted in a run-off in excess of rainfall.

the peak run-off of those storms during 1918 to 1923 which were of general distribution or concentrated near the center of the water-shed. Again, it may be noted in Fig. 36 that storms the centers of which occurred in the lower part of the basin produced higher rates of run-off than the average, and those storms which occurred in the upper part of the basin (which had been drained) produced lower run-off than the average. The black symbols, representing the pre-drainage flood peaks, fit in very well with the general system of post-drainage points. If any difference at all exists, pre-drainage storms have produced higher peaks than post-drainage storms of the same intensity, although the storms are so few in number that this difference can hardly be considered to be of significance.

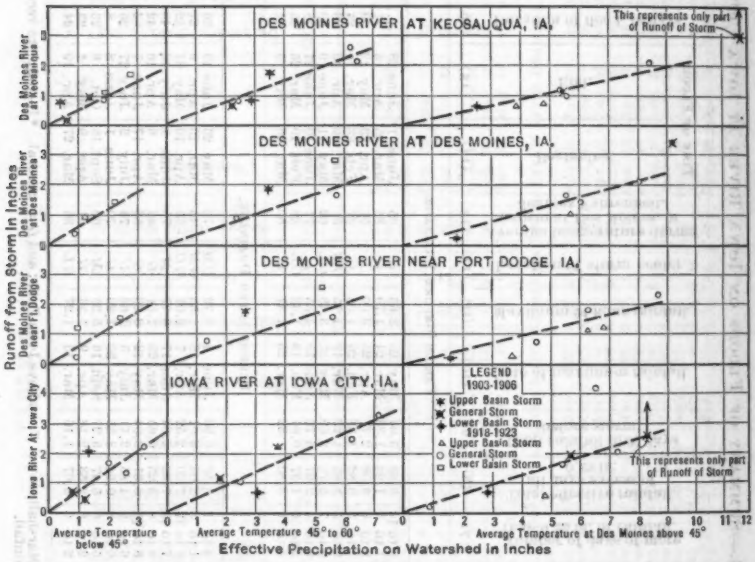

FIG. 35.—RELATION OF PRECIPITATION TO RUN-OFF ON IOWA AND DES MOINES RIVERS DURING HIGH-WATER PERIODS IN 1903-1906 AND 1918-1923.

It remains only to describe the last and most sensitive test applied to the stream-flow records in this attempt to detect some appreciable change caused by agricultural drainage. This test was applied to the curve of diminishing flow following the peak of the storm, during those days of high water when there was no additional precipitation received on the basin. If within a given basin the surface storage has been decreased by drainage in such a fashion that the water is carried away with greater rapidity, the daily rate of run-off should decrease at a more rapid rate than that existing when the ground possesses a higher retentive value. The daily decrease in discharge at given stages of the stream, when undisturbed by subsequent precipitation, was averaged for all floods previous to drainage and compared with similar values

subsequent to drainage. The results are shown in Fig. 37. It is natural that the points should scatter somewhat, due to the difference in depletion rate caused by the location, distribution, and season of the storm, but it is significant that the black symbols representing the pre-drainage conditions show no appreciable difference from the open symbols representing the behavior of the stream subsequent to drainage. Thus, no appreciable difference in the retentive value of the water-shed in so far as it affects the stream flow is revealed by this analysis.

Certain storms which were of unusual intensity, and others which were concentrated over the land which experienced most of the drainage operations, may be of particular interest, and are shown in Figs. 31 and 38. Storms

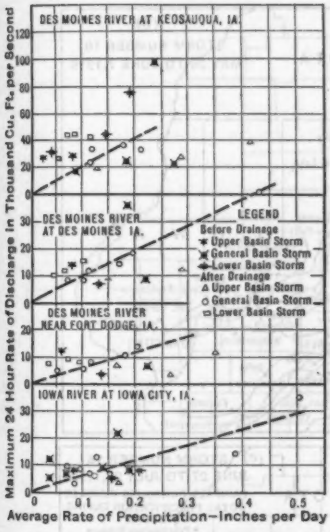

FIG. 36.—RELATION OF PRECIPITATION RATE TO MAXIMUM DISCHARGE OF DES MOINES AND IOWA RIVERS, 1903–06 AND 1918–23.

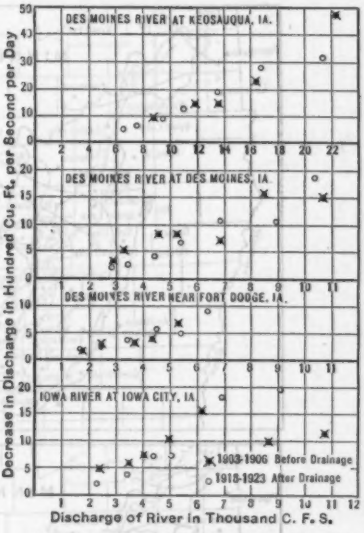

FIG. 37.—RATE OF FLOW DEPLETION, DES MOINES AND IOWA RIVERS.

differ as much as human faces—there are no two alike in amount, location, and time of occurrence, hence no direct comparison is possible between particular storms in the periods before and after drainage. The storm occurring in May and June, 1903, is remarkable in the amount and general distribution of its precipitation, while the storm of May and June, 1918, produced an unusually high rate of rainfall over a large area within a few days.

This exhausts the writers' ingenuity in trying to find a way to detect a measurable effect of drainage upon flood run-off. It may be of interest, however, to note in conclusion that the largest known flood in the Des Moines River Valley occurred in 1851. The flood which reached a peak on June 1, 1903, almost equalled this flood in stage, but since 1903 no flood has occurred which has attained the height of these pre-drainage floods.

FIG. 38.—DRAINAGE AREAS OF DES MOINES AND IOWA RIVERS, SHOWING DISTRIBUTION OF RAINFALL IN STORMS NOS. 9, 10, AND 16.

Likewise, the maximum known flood in the Iowa River Valley occurred in 1851, and again in July, 1881, a torrent only slightly smaller passed down the river. The flood that occurred in June, 1918, in the post-drainage period, attained the highest stage recorded since stream measurements were started, but it was exceeded in magnitude by both of the pre-drainage floods of 1881 and 1851.

DISCUSSION

J. P. SNOW,* M. AM. SOC. C. E. (by letter).—Since before 1850, the levees along the Mississippi River have been added to, strengthened, and perfected. This has resulted, of course, in compelling more and more of the flood waters to flow in the river channel. Before levees were built, any rise above bank level distributed itself over the surrounding territory, resulting in low gauge readings along the river. Records show that flood readings at Natchez, Miss., were:

1882	47.8	1913	52.6
1892	48.1	1916	53.6
1897	49.8	1922	55.5
1903	50.4	1927	56.6
1912	51.4		

These were maximum floods for the past fifty years. The increases in height were due, without doubt, quite largely to the continual improvement in the effectiveness of the levees. The 1927 flood, in spite of the vast volume of water lost through the various breaks, was so high at New Orleans, La., that an artificial crevasse, involving large reparation payments for damages, had to be made to save the city. Further efforts to control maximum floods by levees alone would seem to be absurd. In 1927, if there had been no breaks above Baton Rouge, thereby forcing all the great flood to follow the river channel, the levee system of its lower course would have looked puny indeed.

The normal flow must be held in the present channel of the river to serve the cities and landings along its banks. The surplus of flood flow, above what can be safely carried by the present levees, perfected to a consistent standard, should be taken off by adequate spillways at strategic points. These points are naturally at the mouths of large tributaries, where a great influx of water seeks to enter the river.

The first large tributary north of the Gulf is the Red River, with its branch, the Black River. This is the natural place for the principal spillway. A large basin should be provided there, making use of old bayous, with a substantial spillway with stop-logs along its southern border. The crest of the spillway should be above the level required for satisfactory navigation in the river during the summer season and below the gauge reading at which the present levees, after repairs, can carry the flow safely to the sea; perhaps but little above what is known as bank-full for the general run of the river. It should be long enough to prevent a flood from rising so high as to seriously endanger the levees of the lower river. Perhaps a height of 3 ft. below the tops of the levees may be assumed as a proper summit of flood from which to compute the length of the spillway. It should be designed to carry, at maximum flood, two-thirds of the water at this point seeking transportation to the Gulf, leaving one-third for the main river to handle.

The spillway must be built as a mere overflow without abrupt fall on the down-stream side. Undercutting at the toe of the masonry must be prevented

* Cons. Engr., Boston, Mass.

by sheet-piling properly protected against decay. The Atchafalaya Channel must be cleared to carry the flow from the spillway without obstruction. Width in its upper course is necessary, rather than depth. The slope from the spillway to Grand Lake will be much steeper than that of the main river, and protection against undue scour must be provided. This will be particularly needed for a distance below the spillway. No abrupt fall should be allowed. A particularly wide channel is advisable between Grand Lake and the Gulf, so that the lake will be practically an arm of the sea.

If levees are needed along the Atchafalaya it may be best to construct drainage canals to take the tributaries on each side to Grand Lake rather than to build levees along these side streams. The work of construction is simple in detail throughout, but its magnitude will call for a great expenditure. Its probable ultimate benefit, however, justifies its cost.

This by-pass should hold the flow at a level that will make the river safe as far north as Vicksburg, Miss., at least, and it should prevent overflow of the Red and Black Rivers if their levees are in order. It would seem justifiable to ascertain what the by-pass would do in a flood toward ameliorating the conditions in the upper section of the flood area before attempting further work. If necessary, another spillway could be constructed near the mouth of the Arkansas, or possibly opposite Greenville, Miss., and a channel cut *via* various bayous and creeks into Black River and thence to the Red River Basin. This spillway would probably lower the main river enough, so that the St. Francis Valley would be made safe, and danger would be prevented on the east side at Mound Landing and other points.

The objective is to run off the top of the flood, so as to render the present levee system safe for the main river below Cairo, Ill. Hydraulic data collected during the last seventy-five years should be sufficient to enable computations to be made for the safe design of the structures contemplated herein.

FRANK B. MALTBY,* M. AM. SOC. C. E.—The problem of flood control as a whole is discussed in this Symposium. Other problems treated are the effect of rainfall and run-off on the stage and discharge of the river, forestation, reservoirs, levees, and spillways and auxiliary channels as means of controlling the floods, so as to prevent a repetition of the disastrous results of the record flood of 1927.

All the papers have been prepared by men of experience, of close association with the existing conditions, and with access to an enormous amount of data that have been accumulated by the Mississippi River Commission and others. The sole object of this wide discussion was not simply to add to the available fund of engineering information, nor for the benefit of engineers only. The whole country is vitally interested in the problem, and is looking to the Engineering Profession for a solution.

Engineers are seldom unanimous in their opinion on any subject, and there has been a wide divergence in the ideas expressed on this one, but from discussions of this kind it should be possible to arrive at a consensus of opinion of value to those responsible for the enactment of necessary legislation.

* Cons. Engr., Norristown, Pa.

It is fortunate that, thus far, prominent engineers have led in the discussion of this problem as it has been presented to the public. Especial reference is made to the address of General Jadwin delivered at the Flood Control Congress at Chicago, Ill., in June, 1927. It has so often happened that such conventions have been influenced by poor advice, and have adopted unwise resolutions that have had considerable weight with the uninformed. Engineers have often been content to sit back, shrug their shoulders, and remark that the whole scheme is impracticable. How much better and more valuable it is to lead and to guide than to follow and to criticize!

When it is remembered that the Mississippi River, below the mouth of the Red River, drains from 40 to 50% of the entire area of the United States, and that the alluvial valley of the river contains about 19 000 000 acres of fertile land subject to overflow if unprotected, the assumption that the problem of flood control on the Mississippi is a National one, is well warranted. The whole matter assumes such large proportions and affects such large areas that the solution and the control can only be handled by the Federal Government.

Viewing all the vast and extensive work done by the Mississippi River Commission during the last forty years, it should be particularly remembered that not the least valuable are the accurate, detailed surveys and maps that have been made, the establishment and maintenance of numerous gauges and records, the extensive discharge measurements made at various stages, sedimentation studies, and the collecting and recording of all kinds of data concerning the river. The speaker doubts if there exists anywhere in the world any more complete and accurate information of any river than that collected by the Mississippi River Commission. The voluminous records have made possible the scientific determination of the various features of locating and building levees, bank protection, dredging, etc.

The speaker wishes to advocate the Commission plan for the work of such magnitude as is being done on the Mississippi River, and to make a plea for the continuance of such a form of control of the work to be undertaken by the Federal Government. It is believed that a mixed commission of seven members, as well chosen as the members of the Mississippi River Commission have been thus far, can handle a problem of such magnitude much better than it can be done by any other form of control. Every one appreciates the advantages of centralized control; but this is too big a project for any one man even with the assistance he might command. The membership of the Commission may change, and it does; but all the members do not change at the same time. This results in a fixed and continuing policy which is not affected by each change of administration.

The economic problem of the Mississippi River covers various features, that of providing for the needs of navigation at low stages, prevention of erosion of the banks, as well as control of the flood waters, etc. The speaker will confine his discussion to the latter feature. Floods are caused by rainfall and run-off, as has already been discussed. No man has any control whatever over the rainfall and its distribution, and, consequently, engineering efforts

can only be directed to a proper and reasonable control of the floods after they have reached the river. The only control necessary or desirable is some method of preventing the loss of life, overflow of valuable and fertile lands, railroads, highways, cities, and towns.

In speaking of the control of floods after they have reached the Mississippi River, it is proposed to include that part of the river below Cairo, Ill., or its vicinity only. This limitation will not meet the approval of engineers who feel that the flood solution lies in the construction of reservoirs near the head-waters of the streams. It may be that the construction of reservoirs on the Allegheny, Monongahela, Miami, Upper Mississippi, and other rivers, may be warranted for local control, but it is believed that it has been conclusively demonstrated that any flood control in the Mississippi River delta by reservoirs would necessitate their construction in the delta itself, or immediately adjacent to it. These reservoirs would require such enormous areas to be condemned and abandoned as to make the plan impracticable. The problem is to save from overflow the 19 000 000 acres in the delta, and the States can not give up 4 000 000 or 5 000 000 acres for reservoirs, if it can be done in some other way.

The papers on spillways, auxiliary channels, and their various features as means of flood control are most valuable and interesting. The methods outlined may assist to some extent, but the speaker is firmly convinced that the real solution lies in the construction of adequate levees properly located. Engineers are familiar with the history and growth of the levee system. Their construction has been going on for many years, and the acreage of land protected has steadily increased with the growth of the system. To the present time practically the only method used for controlling the floods, or protecting the land from overflow, has been that of building levees.

When the disastrous flood of 1927 came, a cry went up that the levee system had failed, and that some other solution of the problem must be found. The levees have not failed but, on the other hand, inadequate as some of them were in size, they did, in 1927, protect one-third of the area subject to overflow, and the protected area contains a much larger percentage of cultivated land than that flooded.

One might as well contend that, because costly roads have failed under the increasing demands of automobile traffic, the public should seek some other means of transportation. Instead of abandoning the automobile, people build better roads; so instead of abandoning the levee system, better levees must be built.

It is interesting to note that only one break occurred in Government levees built to standard height and cross-section based on the flood of 1912-13. It does not seem that the engineers made any very great error in designing the levee system as proposed. It is true that occasionally a fire-proof building burns, or a bridge or a dam fails, but that is no reason why every one should stop building fire-proof buildings, or dams, or bridges. Build them better and stronger; also, build better levees if necessary.

The greater part of the area inundated by the failure of levees was caused by breaks in those built by State or local authorities, and largely on the tributaries, where they were not built to the standard cross-section. This statement is not made in criticism of the local authorities or their engineers as they did the best they could with the money available. It only emphasizes the fact that the jurisdiction of the Mississippi River Commission should be extended to embrace the construction of levees on the tributaries as far at least as the protection of the Mississippi delta demands.

There is no desire on the part of the speaker to enter into a discussion of the size, height, or cross-section of an adequate levee. These are details, which can safely be left to those in responsible charge of their construction. It is conceded that provision for a flood, even one somewhat higher than that of 1927, should be made, and that the present standard cross-section is somewhat too small, as in some places the line of saturation falls outside the base on the land side. This results in greatly increasing the cost of maintenance, because a leaking levee must be watched continually as it is liable to fail at any time. The section should be such that in any given material the line of saturation should be always within the base, with water against the levees to within 1 ft., at most, of the top.

The speaker does not wish to be understood as advocating levees, and levees only, as the entire solution of the problem of flood control on the Mississippi River. In determining the grade line of the top of the levees, due consideration should be given to the economy of construction. It would seem hardly necessary to build levees that will provide for the extreme floods that may possibly occur once in 50 or 100 years.

For such unusual floods it may be, and probably will be, more economical in cost, and provide greater safety, to build controlled spillways that will take care of the floods above a predetermined elevation. The time of arrival and height of the crest of the flood will be known and can be provided for by the manipulation of the spillways. Instead of spillways with mechanical control, those of the automatic siphon type might be used. The latter type has proven very satisfactory in operation on a smaller scale, and is worthy of very careful consideration. In any event they should be of large and ample dimensions and of sufficient size to prevent the flood in the channel between the levees going above a predetermined height, even with a discharge as great as may be thought to be possible. A comparatively small acreage of land would be flooded by these spillways and that for a short time.

Owing to the universal use of automobiles and trucks and the scarcity of boats, it would seem desirable to construct a highway on top of the levees to permit ready access during high water for repairs and inspection. Maintenance of the roads would result in more careful inspection of the levees.

The only feature of this discussion that the speaker wishes to emphasize is that of levees. It is believed that they furnish the only adequate method of controlling the floods of the Mississippi. It may be that they can be supplemented to a certain extent by spillways as mentioned, but the main reliance must be found in the levee system.

Remember that the levees protected 6 000 000 acres of fertile land during the recent unprecedented flood. The work done and the money expended on levees to date is not wasted, but will be utilized as a part of the system to be completed to the dimensions that experience has found necessary. Control by other methods, at the best, is problematical. Engineers know the weak features of the levee system and can correct them.

MORRIS KNOWLES,* M. AM. SOC. C. E.—The Ohio River is an important tributary of the Mississippi and it frequently contributes disasters to the lower river. It also has problems of its own, some of which are of local character and have already been solved, notably at Dayton and Columbus, where one can see the works of construction resulting from thorough study.

The head-waters of the Ohio River are at Pittsburgh, Pa., where the Allegheny and Monongahela Rivers unite. At this place, there still remains unsolved (so far as construction is concerned) and lacking in determinative opinion, except from study, "the most important [flood problem] in the United States, in which a single city is the chief interest concerned and in which the chief reliance for protection so far proposed is an extensive reservoir system".†

First impressions of the papers and discussions in this Symposium indicate a note of discouragement. It has been stated that if the present levees had held, the recent flood would have been 6 to 8 ft. higher at several places between Helena, Ark., and New Orleans, La. The Mississippi River Commission reports that a discharge 25% greater may be expected as a probability and that to care for such a flood, within "levees only" would require an increase in present levee heights of from 7 to 20 ft., in the stretch of river to 900 miles below Cairo, Ill.

It is further stated by Colonel Townsend, that:

"Any project which contemplates increasing the heights of existing levees at certain localities about 15 ft., not only affects the item of cost, but raises the question of the practicability and feasibility of maintaining such a levee line after it has been constructed."

* * * * * * * * * *

"* * * no matter how much money is expended on the levee line, it will still be at the mercy of the combination of a muskrat, a dark night and a careless levee inspector. * * * To try to construct a levee line which will resist the river flow under all the imaginary conditions which now fill the public mind, is to attempt the impossible".

Again, it is said by Colonel Kutz, that:

"* * * an 1 800-mile levee line of the height necessary to contain such a flood, would carry with it a menace to the people living behind it that should be avoided if practicable".

At the same time, few, if any, words of encouragement seem to be forthcoming about any of the alternative methods of substitution or means of alleviating conditions and lessening the volume of flood waters to be restrained by such a levee line. Certainly, if these are the facts, then the people of the

* Pres. and Chf. Engr., Morris Knowles, Inc., Pittsburgh, Pa.

† "Flood Control," by the late H. M. Chittenden, M. Am. Soc. C. E., *Transactions*, Inter. Eng. Cong., San Francisco, 1915 (Waterways and Irrigation), Paper No. 30, p. 110

Lower Mississippi Valley should be informed frankly and told that nothing will avail against the greater catastrophe that may and probably will happen. Common honesty and fairness dictate that they should not be lulled into a sense of false security.

The speaker believes, however, that all is not as dark as it is colored, and that the knowledge, training, ingenuity, and research of the American engineer will not falter at the problem. Granted the first three, then research must be brought into action. Mr. Morgan has well argued, based on long and familiar experience with such problems, for the efficacy and necessity of information, resulting from long-time study of an accurate and detailed nature. As to the opportunity for this, as distinguished from the need of haste, people are assured that by 1928 the line of levees will be re-established to the full extent at which they existed prior to the flood of 1927. It has been stated* that:

"Our lines of defense will be closed before the next flood season. * * * Although not entirely completed to standard grade and cross-section, it would seem that, by all the laws of probability, we should be reasonably safe until the revised project can be started."

If this is true, then why all the haste to spend fabulous millions until sufficient data have been received on which to base such large expenditures? American engineers have a real duty to educate the public so as to prevent a stampede of Congress on this important question, to the end that no unwise conclusions be reached, under the false urge of haste, which may burden the country with an erroneous policy for generations to come.

Pittsburgh Problem.—The speaker has been asked to discuss the Upper Ohio situation. This problem and the proposed solution, the reasons for delay, the progress to date, and the possible effect on the lower river have all been subjects of misconception and misunderstanding.

At times, the flood flows from the Ohio River Basin may be an important contribution to great disasters on the Lower Mississippi. While the drainage area is only one-third that of the Missouri (the great tributary of the Mississippi), the rainfall is normally about the same over this area as upon that of the greater river. It frequently contributes one-half the flow to great floods in the lower river. It is possible, also, for its outpouring to synchronize with that of the lower river, according to Dr. Frankenfield.

Western Pennsylvania, including the Allegheny and Monongahela River Basins and the upper portion of the Ohio, together with their tributaries, have been visited by disastrous floods for half a century, culminating in three such events, in 1907 and 1908. The flood of 1907 is the greatest one of which there is any record. It is possible, however, by certain combination of circumstances, distribution of rainfall, degree of saturation of the soil, and conditions of the surface, to have a flood at Pittsburgh at least 10 ft. higher than that which was so disastrous in 1907.

In twenty years, including 1908, the damages at Pittsburgh have been estimated to amount to $17 000 000, of which $6 500 000 was for the three floods of 1907 and 1908. In 1907 the total damage in the Ohio Valley was about

* *Engineering News-Record*, October 20, 1927, p. 636.

$100 000 000. While the streams are reasonably well constricted and the downtown district of Pittsburgh, on both banks of both rivers, contains only about 3 000 acres, about 1 600 acres are affected by large floods. This area includes 100 office buildings, 17 miles of main-line steam railroad, 33 miles of streets, and 7 bridges rendered inaccessible.

Effect of Encroachments.—Statements are constantly made, even in professional circles, but sometimes without adequate basis, that Man's encroachment has been an active agent in increasing, intensifying, and even in producing recent floods. Care should be exercised in such discussions to differentiate between encroachments by embankments in wide alluvial valleys (as they have been described), and those in narrow stretches, restrained by comparatively high elevations on either side, such as exist in many of the upper tributaries. It is, of course, true that floods have been more disastrous in recent years, but to a large extent this may have been because of the greater monetary value of improvements within flood areas, due to the extension of Man's activity. In some cases, of course, encroachments do intensify, floods above such a situation. However, where such restriction is made, it is often possible—in rectifying, straightening, and narrowing the streams—to make the flood discharge of the same capacity and sometimes even greater, by reveting, smoothing, and straightening the banks. Thus, the advent of human occupancy may, in some instances, and can, in most situations, be made not to intensify but to lessen flood troubles.

Organization of Pittsburgh Flood Commission.—Realizing the adverse effect of these occasional visitations and, for the purpose of studying the problem, the business interests of Pittsburgh (acting on suggestions of local engineers through the Chamber of Commerce), created what was called the Pittsburgh Flood Commission, in 1908. This commission included representatives of Allegheny County, the City of Pittsburgh, manufacturing and industrial companies, and the transportation interests. There was a goodly representation of engineers and these were grouped and designated as the Engineering Committee.*

In addition to the question of securing accurate information (by surveys and otherwise) upon which to base an intelligent consideration, the Commission devoted considerable time to studying the effect, both physically and financially, as to the floods which had occurred from time to time. It may be said frankly that the problem was approached, in the first instance (as such problems usually are), with a desire to relieve the local situation. It was thought that the raising of the street and land levels near the rivers, with the possible building of walls, or other barriers, along the water-front, would be of immense benefit. As a result of this discussion, in the subsequent year of 1915, about $500 000 was spent on such regrading work by the City, public

* There were nine engineers, seven of whom were members of this Society, three of whom, Messrs. Swenssen, Wilkins, and Long, have since died. The members of the Engineering Committee were: E. K. Morse, M. Am. Soc. C. E., *Chairman*, the late Emil Swenssen, M. Am. Soc. C. E., *Vice-Chairman*, the late William G. Wilkins, M. Am. Soc. C. E., *Vice-Chairman*, George S. Davison, Past-President, Am. Soc. C. E., Paul Didier, M. Am. Soc. C. E., the late S. C. Long, Gen. Mgr., P. R. R., Julian Kennedy, Morris Knowles, M. Am. Soc. C. E., George M. Lehman, M. Am. Soc. C. E., Secretary and Executive Engineer, and A. B. Shepherd (resigned), Superintendent, Pittsburgh Bureau of Water. (Mr. Didier died May 19, 1928, since this discussion was written.)

utilities, and private owners. Many streets in the low areas of the city were raised and re-arranged to considerable advantage to their use for business purposes. The occasions and the extent of flooding were diminished thereby.

Local Solution not Sufficient.—However, it soon appeared that it was quite possible that such local or selfish consideration would not solve the problem as completely as it should be undertaken. Such restraining walls might have slight effect on some other places near and above the city, but would not materially lessen floods below the city. In addition, a little consideration showed that, at Pittsburgh, as at many other places, while there is too much water at certain seasons of the year, there is too little at other times. This condition sometimes renders the banks of the river and the bed unsightly, diminishes the navigable capacity at times of low stream flow, and increases the injurious effects of pollution and contaminating influences. Dilution has been found to be an important factor in the power of bodies of water to take up and render foreign substances innocuous.

As a natural result of such realization, the Pittsburgh Flood Commission early began to study the possibilities of lessening the large volume of flood waters which comes down to the city at certain times, and increasing the flow during low periods. This naturally meant a study of the drainage area, which is about 19 000 sq. miles, of which 11 580 sq. miles is on the Allegheny and 7 340, on the Monongahela; and 20% of this area is within Pennsylvania; it also comprises about 10% of the Ohio drainage area tributary at Cairo.

Reservoir Possibilities.—By careful surveys and studies of the area within the water-shed of the Allegheny and Monongahela Rivers, it was found there were forty-three possible reservoir sites, which were considered practicable and worthy of intensive investigation. More could have been found if such study had been conducted a half century ago or even a generation prior to this. It is possible that not all of these, found in 1908 and 1909, will be considered now as feasible sites. Thus, does Man by his own works and extensions, sometimes render the problem of conservation more difficult.

Careful and selective study of the availability, effectiveness, and efficiency of the various suggested reservoirs revealed that, of these forty-three, seventeen would be practically and sufficiently effective to take care of all the floods which had ever been experienced at Pittsburgh, except that of 1907. This flood would have been reduced about 8 ft., the average reduction being 10 ft. for all floods. Additional important considerations are, that the use of such reservoirs would not only have reduced flood stages, but would have increased the low-water flow to a measurable extent; the Allegheny low-water discharge would be increased three times, and that of the Monongahela six times.

Each flood since 1900 has been analyzed as to its time of occurrence, duration, and intensity, and the effect that would be produced by each of the seventeen reservoirs proposed on the two water-sheds. Of course, there is no guaranty that floods would occur in the same situations, to the same extent, or with the same periodicity as to extent of time or intensity. However, that which has occurred through several years of recorded

history is quite persuasive as to the probable climatological, meteorological, and physical conditions that are likely to occur in the future. Periods of high water in the streams of the western slope of the Alleghenies occur more frequently at certain times of the year, just as it is more often that there is low water in the autumn. Of the floods in the Pittsburgh District, 75% of which there is any record has occurred between December 1 and April 1, and there never has been a flood of any marked intensity during the months of September, October, and November.

Effect on Lower Rivers.—Some ill-considered comments have been expressed as to the probable effect or non-effect of such reservoirs, wherever they may be placed, on the flow of the Lower Mississippi River. The Pittsburgh Flood Commission and its engineers have never made any claims as to such results, because they have not investigated and studied the effect of the reservoirs on the Allegheny and Monongahela Rivers, on the Mississippi floods. They have, however, made some study extending down the Ohio River about 90 miles. The greatest flood on record at Wheeling, W. Va., would be reduced, by this proposed system of reservoirs, from 13 to 14 ft., and the low-water flow would be increased three times its minimum, on the basis of average normal assumptions as to condition of reservoirs.

Below this point on the Ohio River, there are many large tributaries, and it would be idle, with present lack of data, to say what the effect of the upper reservoirs would be on the high and low-water stages of the river below, to Cairo. It is undoubtedly true, however, that for some distance below Wheeling, perhaps even extending to Cincinnati, Ohio, there would be a marked beneficial effect by the construction of the seventeen reservoirs. The most important requirement, however, is that the solution proposed above Pittsburgh, be studied in connection with the topography, possible locations, distribution of rainfall, and stream flow on the important tributaries below Wheeling, so that idle and immature conjecture as to effect will be replaced with sound opinion founded on careful estimate. It may then appear that favorable sites are found, with possible reduction of flood heights and increase of low-water flow, as a result of the use of such reservoirs. If such be found then, in order to obviate possible injurious interference or clashing, in the use of one reservoir system on another, it points to the need of National control, as to operation and manipulation of reservoir outlets.

National Character of Problem.—It is a National question, because that which is done in one part of the country may vitally affect the lives and property of people in another part. This does not necessarily mean Federal appropriation and expenditures alone, for an important National policy is to create that feeling of responsibility as well as action, to the end that payment for benefits shall be assessed upon and paid to some degree by those particularly receiving benefits. It is well to have, however, the conception of National responsibility, as well as unified control, in order that such a system of reservoirs may be basin-wide in influence. The manipulation of the large systems of storage reservoirs on water supply projects indicates too the kind of centralized control that would make such a system effective and

helpful rather than inefficient and hurtful. Perhaps it should be pointed out that the criterion is not the storage of the entire volume of flood flow, but the retardation of that portion of the excess which, added to the flood flow of other tributaries, produces the disastrous result.

It is recognized, in speaking of possibilities on the Ohio River water-shed, that this has been a matter of controversy for several years. Both proponents and opponents have been outspoken in their assertions, beginning with the monograph by Charles Ellet, Jr., in 1853. However, since then, such discussions have related to the usefulness of reservoirs to increase low-water stages for navigation, in preference to the use of locks and dams for such purpose. Now that canalization is about completed, this question need not be considered and engineers can discuss reservoirs, if and when they have adequate data as to their usefulness in restraining floods. It remains, however, to determine the actual facts as a result of careful and conservative surveys. It is hoped that the forthcoming reports,* requested by the President and Secretary of War, as a result of the Mississippi disaster, will throw some light on such agencies and the best location, construction, and effective use of reservoirs in conjunction with that other very valuable agency of restraining the lower river within suitable and safe levees. It is suggested, however, with present meager information as to topographic maps, stream flow, and precipitation at salient points all over the country, that nine months is too short a time to conclude a study on such a project adequately and comprehensively. If stations for measuring rainfall and stream flow were distributed over the country and universal topographical maps existed, it now would be possible to solve these problems much more quickly.

No Conflict with Power Uses.—One frequently hears the axiomatic statement that empty reservoirs are needed for flood protection and full reservoirs for navigation and water power. In these days when considerable attention is being paid to the development of water resources for power, that is, "white coal", one is constrained to look into the question, is there any necessary antagonism? The Pittsburgh Flood Commission and its engineers believe there is not. It does not, however, advocate the building of great hydroelectric plants in connection with the construction of reservoirs for flood protection, unless real economy and efficiency are to be obtained thereby. The need and request for flood protection should not be used as a guise for securing public appropriations for the development of water power not yet needed; but it may be prudent, in arranging for design, that future possibilities of power requirements be not forgotten.

However, although stated head and constant supply of water are needed for power, and although empty reservoirs are the safe criterion for flood protection, it is still asserted that both uses are not incompatible. By this is meant use at a location and site and not as to volume or storage within certain elevations. The explanation is that a certain volume is needed for flood storage, and a certain head is needed for power. There is nothing antagonistic in providing necessary flood storage at that upper portion of

* The report was presented to Congress in December, 1927.

the dam and within the upper reaches of the reservoir above the elevation required for power. It will frequently be found true that the building of the dam, at a slight increased height necessary for flood storage, on top of a dam required for power, will be less expensive than the building of a separate dam, providing the same amount of storage solely for flood protection.

This point is emphasized, because of the misconception previously mentioned and in order that the position of the Pittsburgh Flood Commission may be made plain; that is, it is not opposed to the development of power, where economically wise, on the streams which unite at Pittsburgh. It has always taken the position of welcoming the investment of private capital, where it can be found to be efficiently done, in the production of electric power by the use of flowing water. It is realized that, even if there cannot be an arrangement to build a dam at increased height, there is a valuable retarding effect, due to flow over the crest of a dam as a weir, thus raising the pond level, whenever there is a heavy precipitation or run-off.

As an example of investigation into this subject, the Flood Commission made a study on three proposed sites on the Clarion River, one of the main tributaries of the Allegheny, having a drainage area of 1 260 sq. miles. The total storage proposed for power purposes was 32 000 000 000 cu. ft. It was even in excess, by more than 20 000 000 000, of storage proposed at some similar sites by the Flood Commission for the complete control of the Clarion River. However, such power dams, not built primarily for flood purposes, would not have as complete effect as if those proposed by the Flood Commission were to be erected. However, they would have had an important deterring and retarding effect on most of the flood heights at Pittsburgh during the last quarter of a century. Of almost equal importance was the fact that the constant flow of water from these power dams would increase the low-water flow of the Allegheny River about three times, and this from the development of reservoirs on one tributary only.

Why the Delay?—It is natural now to ask, if this project be so meritorious, why the country has not heard more about it and something of its construction. The answer is plain. The Allegheny River has portions of its water-shed in two States, New York and Pennsylvania. The Monongahela River has its water-shed in three States, Maryland, West Virginia, and Pennsylvania. Reservoir sites have been proposed in several States and affect the flow of the stream in each. It is not possible, therefore, for the city, county, or even the State of Pennsylvania to carry out the project in its entirety, or in the comprehensive way in which it should be undertaken. Such a contemplation should not be undertaken in a small way—it means National action and control. Heretofore, and from time immemorial, the consideration of the Federal Government has related to the use of large streams for navigation only. The question has been how much would the destruction of the forest, the building of a bridge, encroachment on a stream, or the building of a reservoir affect navigation. If seriously and injuriously, then it became a matter of interest to National agencies and, if this were not true, then Congress was not concerned.

Recently, however, there has been a change of consciousness. Perhaps now, with a realization of the 1927 calamity of the Mississippi River disaster, with an appreciation that the drainage area, population, and wealth comprehends about 40% of the Continental United States, for the first time, Congress may have an appreciation that flood control and regulation of streams is a question to be considered within the whole drainage area, from the source to the mouth, and is a problem for National study, construction, and control.

Forest Benefits.—One of the first acceptances of this view happened when the Week's Appalachian Forest Bill was passed by Congress, in 1911, as a result of which many thousands of acres of land have been secured on the Appalachian Mountain Range for the purpose of protecting slopes from erosion and growing forest cover for its many beneficial purposes. The Pittsburgh Flood Commission early joined hands with the National Forest Commission and upheld its work for this purpose. More than $15 000 000 has been spent in Eastern States, of which $1 200 000 has been on the Allegheny and Monongahela water-sheds, acquiring 367 000 acres of land. Ultimately, it is expected that there will be secured more than 860 000 acres in West Virginia, 80 000 acres in Maryland, and 425 000 acres in Northwestern Pennsylvania. Some of the proposed reservoirs are on lands contemplated for National Forest procurement.

Authoritative Review.—It is appreciated that National action cannot come without Congressional approval, based on a report from authoritative governmental agencies. Shortly after the report of the Pittsburgh Flood Commission was issued, it was reviewed by the U. S. Engineer Office, having to do with the question of the effect of the proposed reservoir system on the navigability of the Ohio River. No question was suggested of course that it would in any way supplant, or replace, or do away with the construction of the locks and dams which are now nearing completion. However, it was realized that at several times in years of low flow, there has not been sufficient water in the Monongahela to allow the boats to pass through the locks without letting out water from the upper pools of the river formed by the dams. Such was the rather general experience during the drouth of 1908.

It was found by the Government engineers that, while such a system of reservoirs would be feasible and practicable, the effect on navigation would be little and would not warrant a participation in expense in excess of 10 per cent. More recently, however, with the newer view of National responsibility as to control of streams, another survey and investigation is in progress by the Federal authorities.

This was brought about through the joint action of the State of Pennsylvania and the National Congress. The State of Pennsylvania, at the session of 1919, appropriated $25 000 to the National Government for making a study of reservoirs proposed by the Pittsburgh Flood Commission and their general effect on the flood question. The Chief of Engineers, under date of December 21, 1916, recommended that the War Department be authorized to make such study. On May 31, 1924, a similar bill was passed by Congress, appropriating $25 000 for this purpose, to be used with the $25 000 from the State of Pennsylvania.

As a result, a study has been going on since 1925, under the District Engineer Officer at Pittsburgh, to look into the question of the feasibility and advisability of reservoirs on the Allegheny and Monongahela drainage areas, to control floods, to increase the low-water flow, and the attendant effect on navigation. This examination is most thorough and extended. It is to be expected that study and criticisms of the location and method of arrangement of the reservoirs proposed by the Pittsburgh Flood Commission will now be forthcoming from an authoritative source.

Times have changed and the cost of public works has notably increased. It is well, therefore, that there be this thorough investigation to settle the question. The country may look forward to receiving the report within a reasonable time, perhaps early in 1928. Then, there may be formulated a constructive program of the National Government as to the development of such a system of reservoirs and its effect locally and extending to the lower river. Perhaps it may be relied on as an example of what may reasonably be done in other places, with similar conditions as to topography, construction possibilities, and relative effectiveness; to the end, in conjunction with other protective measures, that disastrous floods in the entire Mississippi Basin may be prevented forever.

CHARLES H. PAUL,* M. AM. SOC. C. E.—The work of the Miami Conservancy District has been referred to many times in the course of these discussions of the Mississippi River problems. It is significant, however, that it is only the retarding basins, or so-called reservoirs, of the District that have been mentioned. One fact seems to have been overlooked, that nearly one-half the construction cost of the Miami Flood Control System went into levees and river channel improvements. In other words, the Miami Conservancy project is a combination of retarding basins and of river channel improvements, the latter feature including the straightening, widening, and deepening of channels, and the construction or reinforcement of levee systems. The evolution of the plan was to accomplish as much by the channel improvement works as was economically feasible, and to rely on the retarding basins to take care of the surplus water that could not be handled within the limits of economy by further enlargement of channel capacities.

It has been stated that the retarding basins of the Miami Conservancy District have had, or will have, little effect on the flood crests of the Mississippi, and there seems to be a tendency to use this as an argument against any use of reservoirs in connection with the Mississippi River control. It might easily be shown also that the levee system of the Miami Conservancy District will have no effect on the flow in the Mississippi, but certainly no one would go to the extent of giving this as an argument against the use of levees on the Mississippi, although that position would be about as reasonable as the other. The point is that the Miami Conservancy plan was designed solely for the control of the situation in the Miami Valley, and it would only be incidental if the work there had any effect one way or the other on the big problem of the Mississippi.

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The Miami Conservancy project is fully effective for the object for which it has been designed (every one agrees to that), and this is so only because the plans were made after a thorough and comprehensive study of all necessary data. If there is one outstanding fact about the work of the Miami Conservancy District it is the thoroughness with which all necessary information was secured and studied before decision was made relative to the final details of the plan. This fact has impressed not only all former employees, but also all those who have studied the details of the plan. The speaker came into that situation after the general details of the plan had been decided upon; therefore, in commenting on the thoroughness of the preparatory work, he is speaking of the work of other engineers and not of his own. About two years were consumed by these preliminary studies, and all the necessary information was secured even though some of it was obtained with great difficulty, and at considerable expenditure of time and money.

The outstanding difficulty in connection with the solution of the Mississippi River problem is to secure all necessary information that has a bearing on its proper solution. Strange as it may seem, the basic information, such as is required for the development of a comprehensive plan, does not seem to be available. There is no possibility that it can be secured in a few months. It will take several years at best to obtain the necessary data and to make intelligent analyses. It would be most unfortunate if the country were to become committed to a plan, and the large expenditures that necessarily would go with it, until after all such information is assembled and digested, and the best possible scheme is developed. There is need, of course, for immediate appropriations to take care of the reconstruction of works that existed before the 1927 flood.

F. G. JONAH,* M. AM. SOC. C. E.—The recent flood in the Mississippi Valley wrought great damage to the railways in that region, and they are vitally interested in any plan for flood control that will minimize, if not wholly prevent, the tremendous losses that they sustain from these ever-recurring overflows.

There are accounts and records of many floods in the Mississippi River, the earliest being that given by Garcilasso de Vega, who wrote a history of De Soto's explorations. This he based on interviews with men who had been with De Soto on his great expedition. The flood occurred in 1543, and reading the account of it is very much like reading the account of any of the recent floods. One observation might be made with respect to that flood, and also to the others up to and including that of 1844, namely, that the forest growth then covered the entire valley, and apparently had no influence in lessening the flood. This is a fact that should be borne in mind when the project of reforestation is discussed.

Each succeeding flood will be, generally speaking, more destructive than the previous ones for the simple reason that there is more valuable property that can be destroyed. In 1882, when the railway companies first experienced

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trouble, there were about 200 miles of line in the delta. At present, there are 7 500 miles, and other developments have multiplied in proportion, towns, cities, improved highways, increased acres of land in cultivation, and a largely increased population.

From the earliest day of railway construction in the delta to the present there has scarcely been a year when the lines have not been subject to trouble from high water, and it appears that major floods may be expected to occur about once in every five years. The floods affecting railways seriously were those in 1882, 1890, 1892, 1897, 1903, 1912, 1913, 1916, 1920, and 1922. Certain lines sustained great damage, but in the flood of 1927, every road running through the valley below St. Louis, Mo., sustained enormous damages and losses. Due to the flood, train service was suspended on lines aggregating 3 000 miles, and in periods of from 10 to 120 days. The physical damage to these lines has been not less than $10 000 000, and in addition there is a loss, hard to estimate, growing out of the interruption of traffic and the general paralysis of business in the flood sections. These sections equal an area of 18 000 sq. miles, covering more than 100 counties and parishes, in several States, which is equivalent to the area of several States.

Unlike other calamities of Nature, such as earthquakes, cyclones, tornados, and tidal waves (which come without warning), a flood in the Mississippi Valley can be predicted, and with a remarkable degree of accuracy. The U. S. Weather Bureau and the Mississippi River Commission have predicted the passage of the crest of the flood and its highest stages at the several important points along the river to within a few hours of the actual time and a few tenths of a foot of the actual heights attained. It is true that these predictions are revised from time to time during the flood, by reason of the breaks in the levee lines or continued and heavy rainfall. Thus, during the recent flood, there was an unprecedented rainfall over the water-shed of the Arkansas River, which brought that river to one of the greatest stages of record. This made necessary the revision of the predictions for the heights attained by the river at lower points, but these forecasts are so reliable that the railways know what to expect and make preparations accordingly.

The work of fighting the water during inundation is done in three general phases:

First.—To take all possible precautions to prevent overflow and to minimize the damage to track and structures when such overflow occurs.

Second.—To remove the inhabitants and as much of their household goods and live stock as possible from the overflow areas to places of safety.

Third.—To restore track and bridges to service and to resume train schedules in the shortest possible time.

The precautionary measures are the patroling and guarding of the levee lines, and building up the weakest spots with sandbags. In many cases the railways recruit large gangs of men for this work. They place them under the directions of the Government engineers and the municipal authorities, and they transport thousands of sandbags, planking, and laborers to those places that require this emergency work.

There are going on, at this same time, other precautionary measures, such as the cutting in of steam shovels for producing rip-rap, which is always needed in great quantities, and the assembly of work trains, extra gangs, pile-drivers, piling, ties, bridge timber, and rail and fastenings. The track is anchored where it is likely to wash off the embankments, and the ends of openings are protected with rip-rap.

The removal of the inhabitants and live stock is undertaken as soon as the danger is imminent. Many of the people remain until their lives are actually in danger. The railways haul them out to places of safety, furnish hundreds of camp cars and box cars for their temporary shelter, haul water and food-stuffs for them, and provide, in many cases, doctors and medicine. After the flood subsides they haul them back to their homes, and haul seed for their planting, and these services are performed without charge. The railways co-operate with the Red Cross and municipal authorities very closely.

After these precautionary measures are taken, and, when the flood is on the lines, the first effort is to maintain communication as long as possible. Train service is carried on many days when the trains travel through water for many miles at slow speed, and then, after they can no longer run, the plan is to get the work trains in just as soon as the water goes down and restore the tracks for safe operation as quickly as possible.

The railway lines operating in that territory have developed a body of high-water fighters, who, by long experience, are capable of doing great work in these emergencies. Many miles of track are kept in service by raising it on slag, rock, or cribbing, securely anchored; but it is impossible for the companies to raise all their track above flood heights. The cost would be prohibitive. The delta country comprises about 30 000 sq. miles, in which there is now 7 500 miles of railway line, and the railway officials have long since concluded that it would be impossible to raise these lines above flood levels, and, were it possible to do so, they would not serve the country as well as they do now on their present elevations, because they are now adjusted to the industrial situation. Therefore, the railway companies rely on the protection of the levees and take their chances along with the other property owners in the affected districts.

Since the 1927 flood, there has been a mass of literature and discussion in the technical journals and the public press, advocating every conceivable method of control. There have been some suggestions of impounding reservoirs or detention basins in the valley itself. It has been suggested that the St. Francis Basin be used for this purpose, and this entire area of 4 000 sq. miles converted into a catchment basin. Some mention was made of the possibility of this as far back as the report of Humphreys and Abbot, in 1861, but at that time it was swamp land covered with primeval forest growth. It was public domain and the lands could have been segregated by the Government, but now conditions are different. This land comprises the most fertile region of Southeast Missouri and Northeastern Arkansas, in fact, the best agricultural portions of those two States. Statistics will show that, while certain counties in these States either lost population, or were stationary in

growth, in the last two decades, the counties in the St. Francis Basin rapidly increased in population. It is now covered with prosperous cities and towns, farms on every 160 acres almost, and it is one of the few spots in the world where wheat, corn, cotton, and rice can be grown on adjacent fields. Within this basin there are 800 miles of railway lines, and hundreds of miles of concrete highways. These lines could not be abandoned under any circumstances, as they are necessary to keep up communication between Memphis, Tenn., and the great region to the west. The cost of raising any of these lines above the suggested reservoir level would be absolutely prohibitive.

It seems that a more reasonable solution for relief in the St. Francis Basin would be to make a controlled spillway below Cape Girardeau, Mo., leading the water into the St. Francis River, putting levees on this river as well as on the Mississippi. This water, of course, would again reach the Mississippi at Helena, Ark., but it could be carried around in this new channel or by-pass much more quickly than it would come down the main channel of the river, a much longer distance, and thus the crest of the flood might be divided or broken.

A part of the flood waters of the Arkansas might be diverted through a controlled spillway to the Tensas Basin and down the Tensas and Black Rivers to the Red River and the Atchafalaya, through which the Red River discharges when the Mississippi is at flood stage. The main channel might thus be greatly relieved by the diversion of these flood waters of certain of its tributaries.

The settlement of the Mississippi River Valley proceeded along the river itself. The towns and villages are built close to the river bank, because when the country was settled the only means of communication was by the river. Traffic was carried by the steamboats, and towns grew up around the steamboat landings. The highly developed farms front on the river bank because the lands are higher there than elsewhere in the delta. The highways are along the river bank, and the railway lines are likewise very close to the river as they run from town to town. In the proposal of setting certain portions of the levee lines back from the edge of the bank, the railway lines will have to be adjusted to new conditions. There is no doubt that many miles of levee have been built too close to the bank and are subject to the scouring and undermining of the river currents.

It is assumed, of course, that New Orleans must be protected at all hazards, and it would seem that the city can be made secure by converting the Atchafalaya into a permanent controlled spillway, opening up another one through Bayou Manchac, Lake Maurepas, and Lake Ponchartrain, and the removal of many miles of levee lines below the city. As emphasized in Colonel Townsend's admirable paper, the present law creating the Spillway Board, prohibits the discharge of overflow water into Mississippi Sound. This doubtless is due to the objection that New Orleans has always had to the discharge of silt into Lake Ponchartrain, but many engineers believe that it would be a very long time indeed before this occasional discharge would effect the lake. The silt would be deposited in Lake Maurepas and in the

surrounding swamps, and by the time the discharge reached Lake Pontchartrain it would be comparatively clear. This Act could be amended when conditions warrant it.

C. S. JARVIS,* M. AM. SOC. C. E.—It has been the speaker's privilege to glean in broad fields, long since harvested, and to discover values among material habitually overlooked, neglected, or discarded. This opportunity resulted from a co-operative arrangement between the Secretary of War and the Secretary of Agriculture, fostered by the Corps of Engineers, U. S. Army, and the Bureau of Public Roads. In the course of this work it has been demonstrated that the earliest and rarest hydrographic records, far too meager at best, are largely lying dormant as an undigested mass, or as detached and unrelated fragments, although they are capable of being assembled and utilized to great advantage. American engineers have been accustomed to think that the longest periods of meteorological and hydrographic record were for about 60 years, when as a matter of fact they exceed a century.

Fig. 39 depicts the relation of run-off to precipitation on two of the water-sheds supplying New York City. The habits of the Esopus area are portrayed from 1906 to 1926, inclusive. The monthly precipitation, which is platted above the axial line, occasionally produces run-off at nearly the same rate; but it lags behind the stormy period as required for percolation, thawing, or concentration. The average monthly precipitation ranges from 3.5 to 4.5 in., and the depth of run-off to be expected during March or April is fully 6 in. Even though the rainfall is greatest during the summer months, the yield is the least, due to the higher temperature, rates of evaporation, absorption by the soil, interception by foliage, and the demands of vegetation during the season of growth. The run-off from the 257 sq. miles of drainage area during the period of record was 31.2 in. in depth yearly (if considered evenly distributed over the water-shed), or 65.4% of the precipitation.

The hydrographs for the Croton drainage area extend from 1868 to 1926. The orderly behavior of the run-off since 1875 would be an excellent argument for reducing or controlling floods by forestation and wise supervision of the water-shed, were it not for such years as 1902 and 1920. The monthly run-off during the spring thaws in each of these years was double or treble the average monthly precipitation, and this could not fail to produce violent floods above the reservoir. The average annual run-off during the 59-year period of record was equivalent to a depth of 22.6 in. over the 375 sq. miles of water-shed, or 47.2% of the average annual precipitation. Only two meteorological stations were maintained for the first few years on this area, but the number was periodically increased, and for the past 20 years nine stations have given daily reports. The maximum annual rainfall recorded for this entire area was 63.76 in. in 1901; the minimum was 37.62 in. in 1880. As might be expected, those years marked the extremes of annual run-off, 34.4 and 11.8 in., respectively. The observed monthly precipitation frequently reaches double the average and rarely if ever attains three times the average for a given month.

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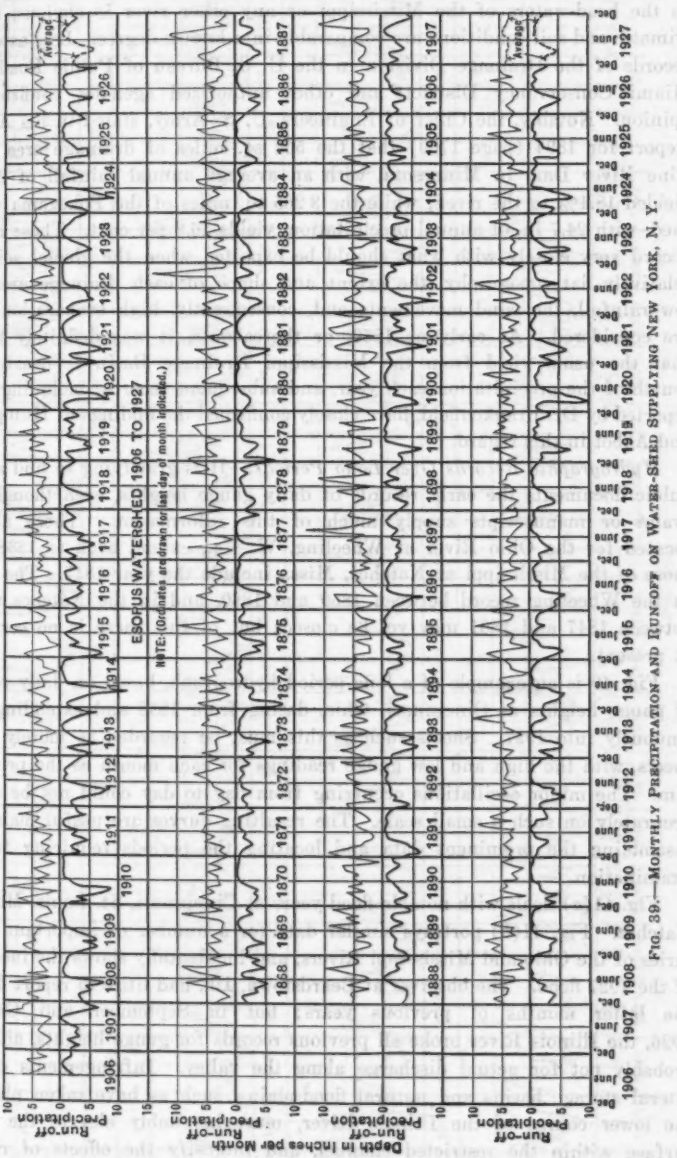

FIG. 39.—MONTHLY PRECIPITATION AND RUN-OFF ON WATER-SHED SUPPLYING NEW YORK, N. Y.

Application of Basic Data to Other Drainage Areas.—The application to the head-waters of the Mississippi or any other river is obvious, if the climatic and soil conditions are comparable in a known degree. Less extensive records of the Drainage Division in the U. S. Bureau of Public Roads, the Miami Conservancy District, and other authorized agencies confirm this opinion. Notably, the Chief of Engineers, U. S. Army, stated in his Annual Report for 1894 (page 1709) that the 562 sq. miles of drainage area above Pine River Dam in Minnesota, with an average annual rainfall of 23 in., yielded 18.4% to the river; while the 3 265 sq. miles of the Pokegama watershed, with 24.7 in. of annual precipitation, yields 16.8 per cent. These results accord very closely with what should be expected when the glacial soil, the relatively flat topography, the extent and shape of each drainage area, the low rainfall, the wind movements and, consequently, high evaporation rates are considered. As early as 1850, or thereabouts, it was definitely known that the usual yield from the Mississippi Drainage Basin is nearly one-fourth of the precipitation each year, and subsequent records, including those reported by Dr. Frankenfield, have closely confirmed the findings of Humphreys and Abbot in this regard.

Hydrographic Records Over Long Periods.—It is gratifying to find among public documents the early records of daily gauge heights, even though diagrams or manuscripts supply much of this information. Those already located for the Ohio River at Wheeling, W. Va., extend back to 1838; and those of the Mississippi at Natchez, Miss., include the year 1817. The break in the Wheeling record between 1850 and 1860, and in the Natchez record between 1847 and 1861 may yet be closed; but of this there is no certainty at present.

Fig. 40 is an example of a long-period hydrograph, based on daily records of gauge heights at Cincinnati, Ohio, dating from 1858 and extending continuously into 1927. Sheets such as this must be regarded as merely index sheets, with the high and low gauge readings for each month as the principal aim. The minor oscillations occurring from day to day could not be shown accurately on such a small scale. The resulting curves are useful mainly in assembling the prominent data and locating the periods requiring further examination.

Fig. 41(*a*) deals with notable flood years at Cincinnati, St. Louis, Mo., and Natchez. Fig. 41(*b*) portrays similar data for a number of important tributaries of the Ohio and Mississippi Rivers, and incidentally shows the inception of the 1927 flood. The observer at Beardstown, Ill., had little to report during the latter months of previous years; but in September and October, 1926, the Illinois River broke all previous records for gauge heights, although probably not for actual discharge along the valley. Infringements on the lateral storage basins and natural flood-plains, such as have taken place on the lower course of the Illinois River, must inevitably elevate the water surface within the restricted channel, and intensify the effects of record-breaking storms such as occurred in September, 1926.

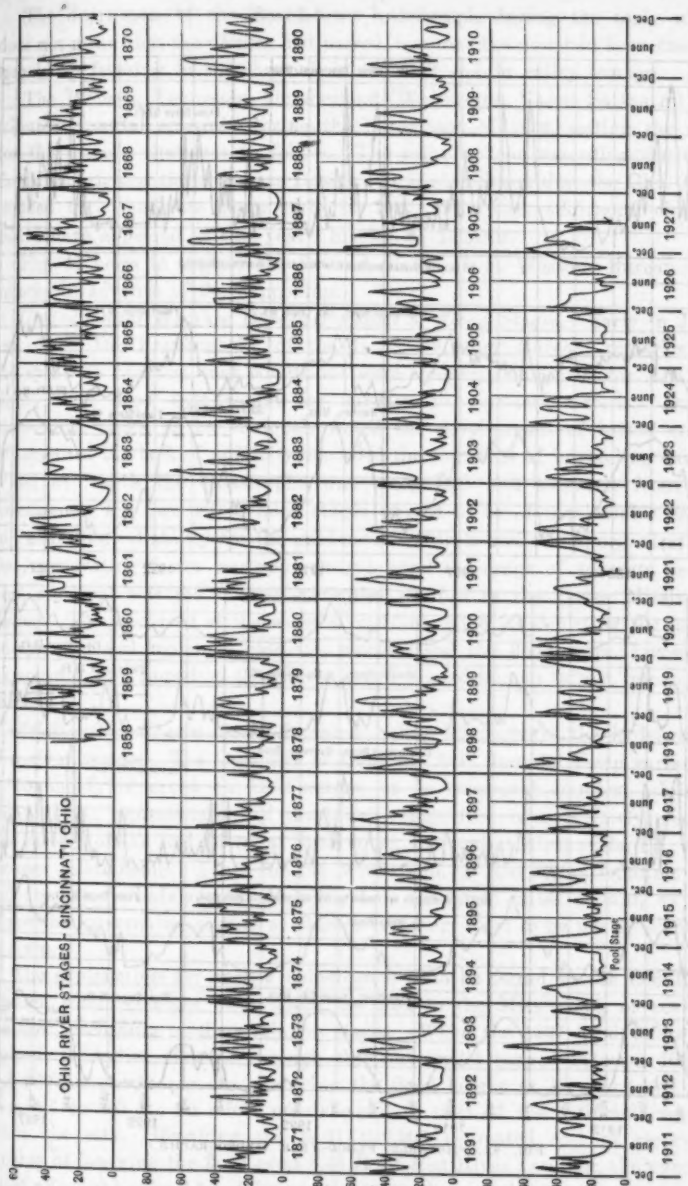
OHIO RIVER STAGES- CINCINNATI, OHIO
Gauge Height in Feet
Pool Stage
1858 1859 1860 1861 1862 1863 1864 1865 1866 1867 1868 1869 1870
1871 1872 1873 1874 1875 1876 1877 1878 1879 1880 1881 1882 1883 1884 1885 1886 1887 1888 1889 1890
1891 1892 1893 1894 1895 1896 1897 1898 1899 1900 1901 1902 1903 1904 1905 1906 1907 1908 1909 1910
1911 1912 1913 1914 1915 1916 1917 1918 1919 1920 1921 1922 1923 1924 1925 1926 1927
Dec. June

FIG. 40.

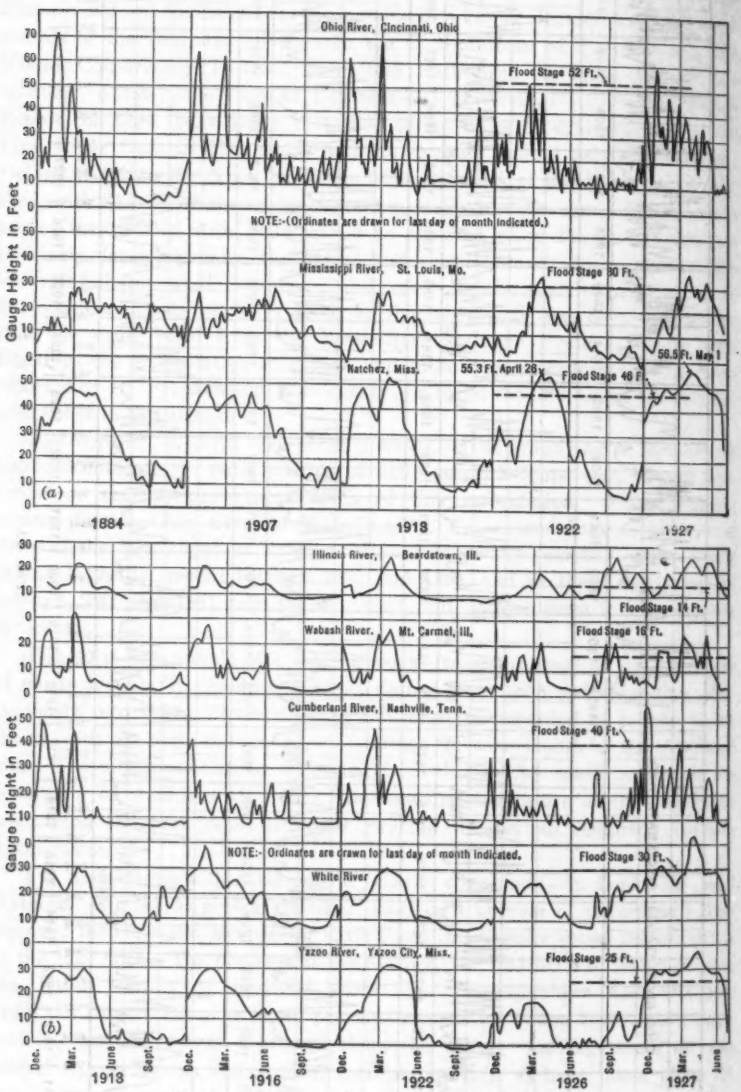

FIG. 41.—NOTABLE FLOOD-YEAR HYDROGRAPHS.

The depression of the Beardstown hydrograph during the early winter does not mean that the danger had passed, but that the flow had been checked, largely by freezing in the channel, in the soil, and in valley storage.

The Wabash, Tennessee, Cumberland, White, and Yazoo Basins all contributed unusual amounts during the latter part of 1926, setting the stage for the tragedy that was to follow. The same is true regarding the Ohio, the Missouri, and the Arkansas Basins. It is significant that the Ohio River receded to a moderate stage early in 1927; otherwise, additional chapters would have appeared in this year's history of disaster.

Fig. 41 helps to explain the forecast of the U. S. Weather Bureau some months in advance of the happening.

United States Weather Bureau Forecast of Flood Stages.—Early in January, 1927, Dr. Frankenfield, of the U. S. Weather Bureau, remarked, in the speaker's hearing, that conditions were shaping themselves for a Mississippi River flood that would far surpass all previous records. During succeeding weeks, as he anxiously noted the development of initial stages, he lamented the lack of publicity accorded the warnings of impending danger. When at length the catastrophe was recognized and accorded front-page prominence, the forecast of flood stages proved to be of incalculable benefit and unfailing reliability for the guidance of patrols and rescue organizations. One aggravating factor was apparently beyond the range of accurate prediction, and that was the intense torrential rainfall in the Lower Mississippi Valley. This occurred at the worst conceivable time for depleting the lateral storage and soil moisture capacity, augmenting the flood wave, increasing its velocity, and impeding the efforts aimed toward human safety, rescue, and relief.

Relationship Shown by Hydrographs.—Fig. 42 portrays the hydrographs of a river system. The Ouachita at Camden, Ark., has the sharp variations so commonly observed on head-waters, or in semi-arid country, while at Monroe, La., representative of the lower valley, the curve is fairly smooth. The records of the Red River, at Shreveport, La., and at Alexandria, La., show a closer relationship. Those for the Mississippi, at Red River Landing, La., and for the Atchafalaya, at Melville, La., nearly 40 miles distant, as well as for the intervening station at Barbre Landing, seem to be nearly identical in form.

The explanation for such a coincidence is to be found in the Old River channel which conveys water from the Red to the Mississippi, or reverses the flow, according to their relative stages. When the 1927 flood was at its highest, the relief afforded through this one outlet ranged between 100 000 and 200 000 cu. ft. per sec., reducing the flood height at New Orleans, La., from 1.7 to 3.0 ft., an effect nearly equal to that of the artificial crevasse below the city. Recalling the $1 313 000 000 estimated cost of reservoirs capable of lowering the flood crest 3.36 ft. at Red River Landing, the effective relief afforded by the Atchafalaya outlet is worth more than has been expended to date (1927) on Mississippi flood control.

According to official current meter measurements,* the discharge of the Mississippi River through or adjacent to Old River channel has exceeded 250 000 sec-ft. at some time during each of the 1912, 1913, 1916, 1917, 1920, and 1922 flood seasons. The same authority indicates that other notable measurements were as follows: 154 000 sec-ft. in February, 1907; 347 000 in April, 1912; 317 000 in April, 1920; 447 000 in April, 1922; and 206 000 in April, 1923.

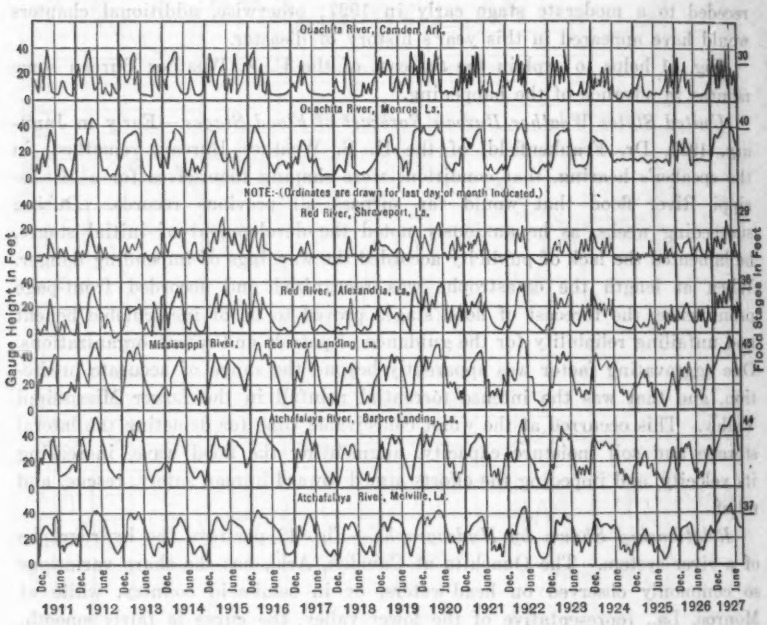

FIG. 42.—HYDROGRAPHS OF A RIVER SYSTEM.

Examples of Relief Outlets and Automatic Spillways.—In spite of the sporadic notions promulgated during the past eighty years for divorcing the Red and the Atchafalaya from the Mississippi drainage, or for closing the Atchafalaya at Barbre Landing against the Red River, this great relief outlet has been consistently maintained and improved by the U. S. Corps of Engineers. Log rafts and other obstructions have been removed, inducing the current to enlarge its own channel until the capacity now exceeds 400 000 sec-ft. for a 50-ft. stage at Barbre Landing. This represents an increase of fully 50% since 1882. Meantime, some revetment and sills had to be provided near Simmesport, La., to insure against the possible depletion or capture of the main river. Thus, doubtless, originated the belief that the engineers were gradually closing the outlet.

* "Results of Discharge Observations. 1838–1922," Pamphlet compiled and issued by the Mississippi River Commission, 1925.

Automatic spillways of the most expensive and wasteful type have operated at intervals ever since the first levees were established; and these crevasses will recur until adequate spillway facilities are provided to protect the levee system.

The Fiction of "Levees Only".—Because the engineers employed in controlling the Mississippi have consistently sought to complete the levees to standard grade before diverting funds and energies to other expedients, they have been regarded as the sponsors of "levees only" as the means of control. The reports of the Chief of Engineers are replete with discussions of divergent views on this subject. Similarly, the report of the State Engineer of Louisiana for 1850, page 39, advocated outlets or spillways; the Annual Report for 1861, page 38, condemns such a proposal. After additional years of investigation the conclusion was recorded* that:

"Outlets of sufficient magnitude to appreciably depress the floods, must either deluge vast areas of our State, as the Ashton and Grand levee or Morganzia, or be most expensive and injurious, as would be the Bonnet Carre and Lake Borgne, and therefore I oppose them, Lafourche and all, and turn for safety to our present levee system."

Despite such sentiments among local residents and officials, it is significant that the main reliance continued, not as "levees only", but levees aided by a great relief outlet and overflow sections or the inevitable crevasse within the uncompleted structure. Furthermore, extensive storage was proposed, accomplished, and operated on some of the head-waters as an aid to flood control and its natural corollary, namely, increased flow during the low-water season.

The "levees only" policy is fiction at best; for actually other means have passed the experimental stage. A relief channel operated throughout American history has convinced most of the former opponents of spillways.

Regarding Physical Limitations.—Try to imagine the 1927 flood confined between levees, 10 or 15 ft. above the completed embankments of recent years, resisting a current velocity of 12 ft. per sec., or 8 miles per hour, double or treble the safe velocity along unrevetted banks of silt; and then return to reasonable measures.

The proposed location of the levees a few miles back from their present location would encounter the same difficulties because of the lateral slope of nearly 7 ft. per mile away from the main channel. Considerations of foundation pressures, structural security, and the safeguarding of communities, all seem to forbid a great increase of levee heights. Unless adequate detention or spillway facilities are provided, the river assumes the mastery and Man becomes the fugitive.

A Résumé of River Habits.—Fig. 43 illustrates a device originated by the speaker for condensing hydrographic data and transforming it into convenient units. Such a system of parallel scales would present on a single page the most essential elements of voluminous records and extensive research, forecasts as to probable future recurrence, and a convenient means of comparison. Thus, the 1927 flood at Red River Landing with a gauge height of 57.5 ft.,

* Annual Rept., State Engr. of Louisiana, 1871, p. 20.

discharged nearly 1 600 000 sec-ft., or 1.3 sec-ft. per sq. mile of drainage area; or at the rate of 1.5 in. depth per month over the entire water-shed. The probable frequency of recurrence, judged by the whole unweighted record, is once per century; or in the light of the river's aggravated habits during recent years and their prospects for continuance, it may better be classed as a 25-year flood. The rating on the Myers scale is 14.5 per cent. Fig. 43 is based on 57 years of continuous record to 1927.

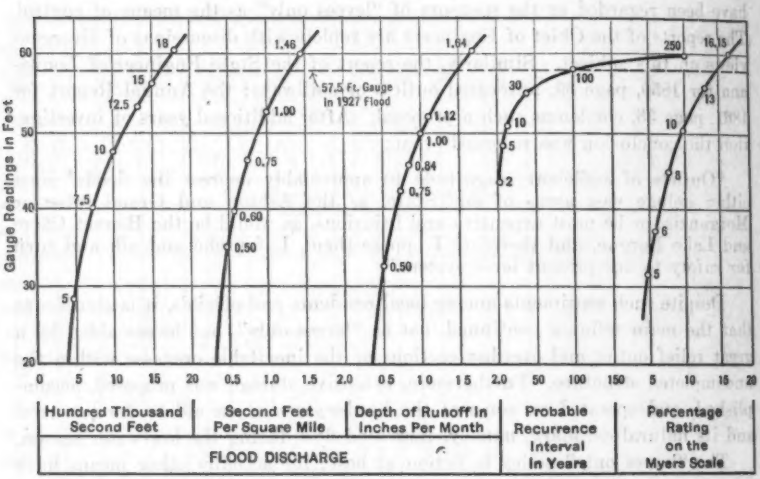

FIG. 43.—ANNUAL FLOOD RATINGS OF THE MISSISSIPPI RIVER AT RED RIVER LANDING, LA.

Using the same general method of analysis in 1923 and 1925, the speaker reached the conclusion* that the rating at Helena, Ark., for rare floods should be 25%, corresponding to a discharge of 2 500 000 sec-ft. He also urged the adoption of spillways to afford relief, and suggested a general re-appraisal of the flood situation to meet the changed conditions wrought by Man. Information disclosed after months of research seems to confirm both the estimate and the recommendations. The main significance of the 25% rating lies in the probability of discharges exceeding 2 500 000 sec-ft. at Red River Landing if the levees hold. The net accretion at this station from below Helena, should be measured approximately by the maximum concurrent contributions from the White, the Arkansas, the Yazoo, and Red Rivers, minus the outflow through the Atchafalaya. As this depends on the stage of the Red River it seems advisable to investigate the feasibility of diversions above Alexandria, into such neighboring drainage as the Sabine River, where local benefits would accrue. Likewise, it has been suggested that, during emergencies, part of the Arkansas River run-off should be conducted southward through bayous and never be allowed to reach the Mississippi.

* "Flood Flow Characteristics," by C. S. Jarvis, M. Am. Soc. C. E., *Transactions*, Vol. 89 (1926), p. 1023, Item No. 947.

Need for Topographic Maps.—The need for topographic maps of this region, to facilitate the location of auxiliary channels and lateral storage basins, or detention areas, was never more imperative than at this time. Fortunately, the 1927 flood-plane can still be traced, and there has been a demonstration of how much spreading is required to dissipate a major flood.

A Graphical Solution of Flood Probabilities.—Fig. 44 illustrates the speaker's method of determining flood periodicity by analyzing the trend of existing records. This has been described in detail* and has thus far satisfactorily withstood tests and comparisons of results with the various approved analytical methods. In addition to being much less laborious and involved, the graphic solution seems to be more readily interpreted. It is to be observed that, for Red River Landing, the same result is obtained whether the gauge readings or the distances above the flood stage of 45 ft. are plotted. For a 400-year period both curves indicate a probable stage of nearly 61 ft., if the entire unweighted record is considered.

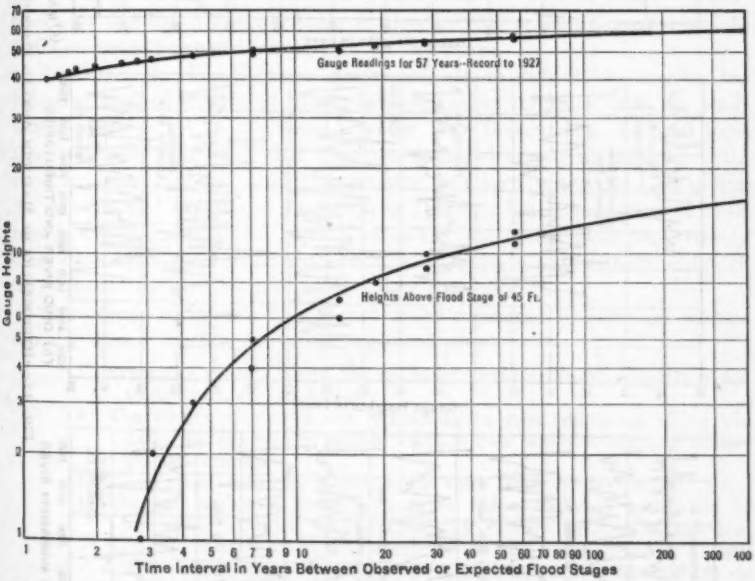

FIG. 44.—TIME INTERVAL, IN YEARS, BETWEEN OBSERVED OR EXPECTED FLOOD STAGES, ON THE MISSISSIPPI RIVER, AT RED RIVER LANDING, LA.

Maximum Annual Flood Heights.—Fig. 45(*a*) represents the maximum annual stages attained on important tributaries of the Mississippi. The unmistakable upward trend of the Illinois River, at Beardstown, has been observed with alarm for several years; this is also true of the Yazoo and other rivers under similar treatment.

* Discussion by C. S. Jarvis, M. Am. Soc. C. E., of paper entitled "Probability of Flood Flows," by F. G. Switzer, Assoc. M. Am. Soc. C. E., *Proceedings*, Am. Soc. C. E., October, 1927, Papers and Discussions, p. 2027.

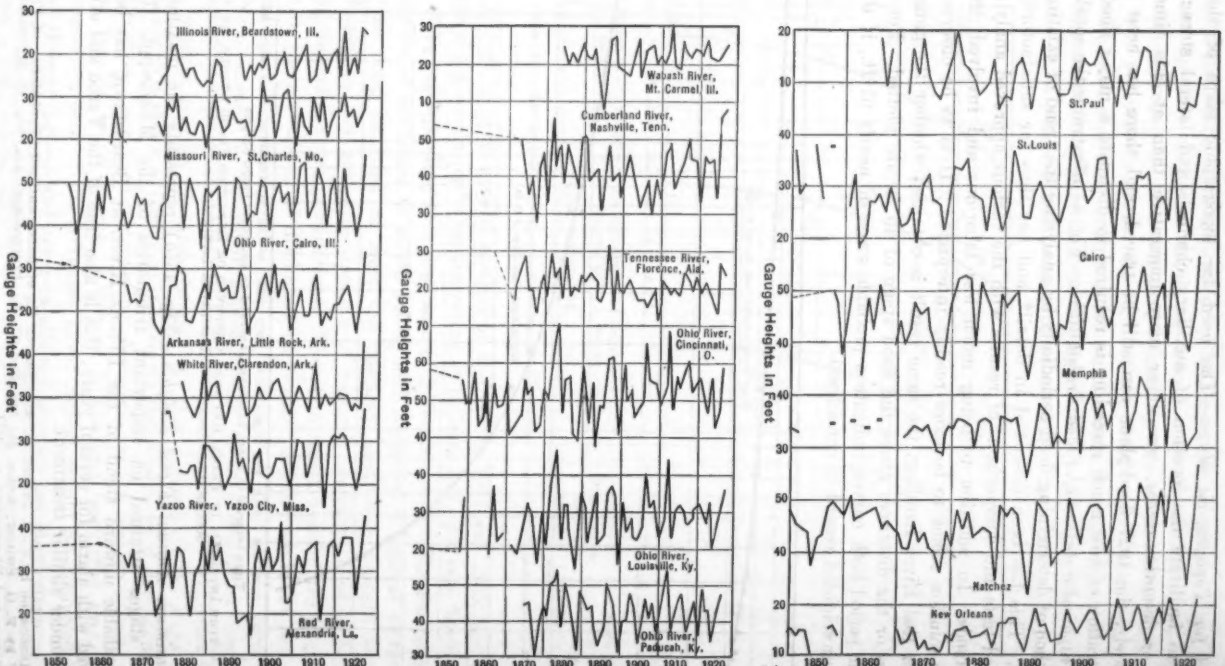

FIG. 45.—HYDROGRAPHS OF MAXIMUM ANNUAL FLOOD-STAGES.

Fig. 45(*b*) indicates that the Ohio River and its principal tributaries are gradually yielding to some form of restraint and regulation. Among these influences, not yet evaluated, but generally under-estimated, are channel improvements, detention basins, and numerous reservoirs operated solely for local protection or enterprise.

Fig. 45(*c*) displays the definite upward trend so characteristic of all Mississippi River stations between Cairo, Ill., and the Gulf. It also shows the neutral tendency at St. Louis, and the evident downward trend at St. Paul, Minn. There has been no progressive diminution of precipitation in this area throughout the period of record; the brief cycles of either sub-normal or excessive rainfall are readily traceable (Fig. 45(*c*)). The record of gaugings by current meter between 1866 and 1917 indicates that the cross-section has remained fairly constant, so that the diminution of flood crests at St. Paul means a decreased rate of discharge, not only apparent, but real. The effective lowering amounts to several feet during the past 50 years. Translated into discharge, this reduction is about 40 000 or 50 000 sec-ft., which, at Cairo, would represent nearly 1 ft. on the gauge.

Regulation by Head-Waters Reservoirs.—It is difficult to ascribe the reduction of flood heights at St. Paul to any other influence than the head-waters reservoir system established by the War Department. So well have these reservoirs functioned that observers doubt whether they have yet withstood a severe test. A similar impression has been entertained regarding the structures of the Miami Conservancy District; but examination of their records will prove that they have been called into action more than twenty times since 1922, and have averted material damage in that district, although the recent storms have attained only moderate severity.

This regulation of the flow is not to be measured by nominal storage capacity alone. Factors too often neglected are:

(1) Reservoir storage above the spillway level; ordinarily nearly equal to that which lies below.
(2) The effects of back-water, extending to great distances along the main valleys and retarding surface flow while promoting underground storage.
(3) Percolation into the lateral areas thus temporarily flooded.
(4) The soil moisture capacity reserved below the dam by reason of the delay afforded by the foregoing factors against inundation.
(5) The lag in movement of water outside the main channel, always subtracting from the flood crest and extending the period as well as the losses in transit.

Due to these factors, the actual retardation accomplished by a reservoir with an adequate spillway is a multiple of the apparent capacity. This has been demonstrated repeatedly on various streams, under conditions and control roughly simulating those of the laboratory.

Salvage of Fragmentary Records.—Investigators have been richly rewarded for applying the winnowing process to old records. The observations at Natchez, extending intermittently from 1770 to 1802, and each year thereafter, are outstanding examples. No doubt, there are other data to

be uncovered in the reports of the Chief of Engineers, U. S. Army; the Weather Bureau; the Geological Survey; State and municipal engineering departments; power and irrigation companies; and occasionally in private files or libraries.

A commendable spirit of co-operation in meeting the immediate demands of various governmental departments recently produced results far beyond original expectations. The foregoing illustrations are representative of a great mass of information hitherto unassembled, but reposing in hundreds of volumes, or in manuscript form.

To accomplish what is required in this broad field, some competent agency must be financed and set to work, with powers commensurate with its difficult task. After the records have been interpreted and co-ordinated their most essential elements should be reduced to graphic form and summarized in some convenient manner for each station, as has been shown on Fig 42.

The Flood-Control Problem Defined.—The problem of flood control is simplified by consideration of the relatively large volume for which provision has already been made in channels and in some form of storage. The relatively infrequent flood stage attained on large rivers, and the few days or weeks that it continues above the danger line, reduce the difficulties proportionately.

Definite, tangible, and positive control projects are needed immediately. These require popular support which cannot be had by antagonizing, but by recognizing, every commendable theory or movement. Therefore, every reasonable means of improving or maintaining soil structure, fertility, and vegetative covering, stabilizing drainage channels and reducing erosion, promoting the passage of surface water to soil storage, and allowing for its orderly release therefrom, should be encouraged as steps in the right direction. No matter how remote and indirect may be their influence, they benefit the locality; they enlist conscious support and sympathy; they combine free-will offerings with taxation toward a solution of America's foremost problem. They bear the same relation to effective flood control as do war-savings, abstinence, and speeding up of industry to the winning of a foreign campaign.

Checking of soil erosion in Alpine regions, for example, seems to be as imperative in Switzerland as the defense of the fatherland; or perhaps it is one means of defense which is worthy of adoption in other mountainous districts included in American problems.

Only one-fourth of the total rainfall appears as run-off in the Mississippi River. This is accommodated without material damage on an average of three years out of every four. The minor portion of the highest known flood is the destructive element, and that is susceptible of control.

Conclusions.—

1.—"The Mississippi needs more room", as aptly expressed by Col. George R. Spalding, Corps of Engineers, U. S. A., after conducting the campaign of rescue.

2.—Its control is a National problem and should be accomplished as the nation would repel an invasion.

3.—Wherever human life is at stake the protection should be positive and enduring. Where only inconvenience and slight damage would accrue from temporary flooding, this means should be encouraged and insured during emergencies such as the 1927 flood, as a means of dissipating the power of the flood. Such automatic relief is as essential to storage basins and river channels as are safety valves for steam boilers.

4.—Levees and other barriers as a first line of defense, assisted by reservoirs and regulating basins, auxiliary channels and spillways, additional outlets and diversion into adjacent drainage areas, all correlated, under intelligent supervision and control, must be utilized in any effective plan for the mastery of the Mississippi River.

EDWARD H. SCHULZ,* M. AM. SOC. C. E. (by letter).—The continued and heavy rains of the fall of 1926 and spring of 1927, brought about an extraordinary flood in the Mississippi River and its tributaries, from Rock Island, Ill., to the Gulf of Mexico. Rains were exceptionally severe on the watersheds of the Illinois, White, and Arkansas Rivers and in the valley below Cairo, Ill. There was much damage to property, much flooding of land, a large loss of farm animals and crops, and complete interruption of rail and highway traffic. Above all, there was acute suffering among the people, although fortunately not a heavy loss of life, thanks to the relief measures of the Red Cross, of the Army—the Quartermaster, Air Corps and Engineers—and of other Governmental agencies.

Protection Against Future Floods.—The Mississippi River Commission, with the approval of the Secretary of War and the President, was directed by the Chief of Engineers to prepare a revised plan of protection against a probable flood on the main river and its tributaries, as far as they are affected by the back-waters of the Mississippi River floods. Public hearings were held, and investigations of rainfall, gauge heights, and discharges have been made with a view of presenting a comprehensive report, a plan, and an estimate for the action of the Chief of Engineers, the Secretary of War, and for presentation by the President to Congress. Committees of the Commission and the District Engineers in charge of local work have studied and reported on auxiliary means of help, such as dredging, reservoir, diversions, and outlets, as well as on the general subject of levees, including greater free-board and larger sections.

Present Levee Jurisdiction.—The Commission's jurisdiction is limited by law, for levee and other protection work, to the Mississippi River below Rock Island, and the tributaries as far as they are affected by Mississippi River floods. The subject of a general survey of all tributaries, as well as of other rivers of the United States, covering floods, irrigation, water power, and navigation, was authorized by Congress, January 21, 1927, based on a report in House Document 308, 69th Congress, First Session.

* Col., Corps of Engrs., U. S. A.; Div. Engr., Lakes Div., U. S. A., Cleveland, Ohio.

Physical Characteristics.—The distances along the Mississippi River and the slopes at low water are as follows:

Rock Island, Ill., to mouth of Missouri River.....302 miles, slope 0.4 ft. per mile.
Mouth of Missouri River to Cairo, Ill.....209 " " 0.6 " " "
Cairo, Ill., to mouth of the Red River .....772 " " 0.35 " " "
Mouth of Red River to the Gulf .....309 " " 0.02 " " "

The alluvial valley begins about 52 miles above Cairo near Cape Girardeau, Mo. From this point to the Gulf the river flows in a valley varying from 20 miles in width near Natchez, Miss., to 80 miles near Greenville, Miss., the average being about 45 miles. Below Plaquemine, La., the valley merges into the fan-shaped delta with a radius of 60 to 100 miles.

Rock is found not far below the bed at St. Louis, Mo. From St. Louis to Cairo, the average depth to rock is about 92 ft., varying from near the bed to 130 ft. below. The general drop is 105 ft. in this distance. Below Cairo, the dip is rapid. No difficulties with rock would occur therefore in any river or flood improvement.

The Water-Shed.—The area of the whole Mississippi water-shed is 1 240 000 sq. miles, or about 41% of the United States. The extreme dimension in longitude is 1 900 miles, extending from the States of New York to Montana, and in latitude 1 400 miles, from Canada to the Gulf. Of the total area, 390 000 sq. miles are arid, 190 000 sq. miles, semi-arid, and 660 000 sq. miles are humid.

The distance from the head-waters of the Missouri to the Gulf is 4 200 miles, from Lake Itasca, 2 475 miles, and from the head-waters of the Ohio, 2 370 miles.

The six natural divisions of the drainage and normal rainfall are as given in Table 35.

TABLE 35.—THE MISSISSIPPI WATER-SHED.

Division.	Area, in square miles.	Ratio to entire basin.	Normal precipitation, in inches.
Upper Mississippi	165 900	0.13	30.9
Missouri River	527 100	0.43	20.7
Ohio River	201 700	0.16	44.2
Arkansas Basin	186 300	0.15	29.8
Red River	90 000	0.07	38.3
Central Valley	69 000	0.06	48.8
Total	1 240 000	1.00	30.1

It will be noted that the average rainfall is about 30 in. During March and April, 1927, 25 in. of rain fell, and in January to April as much as 35 in.

in some parts of the lower valley, more than 23 in. above the normal for those months.

The Delta Basin.—In general, floods on the Lower Mississippi occur from February to May, and low water from August to December. The delta area, normally subject to overflow (except for the levee system), is 29 790 sq. miles in extent. This is equal to about 19 000 000 acres, divided, as follows:

By States:	Square miles.
Illinois	65
Kentucky	175
Tennessee	453
Arkansas	4 652
Mississippi	6 926
Louisiana	14 695

By Basins or Districts:	Square miles.
St. Francis and miscellaneous in Illinois, Kentucky, and Tennessee	6 706
Yazoo	6 648
White	956
Tensas	5 370
Atchafalaya	6 085
Pontchartrain	2 001
Lafourche	2 024
Total	29 790

The flood of 1927 covered a great portion of these areas. The previous large floods occurred in 1828, 1844, 1858, 1862, 1882, 1883, 1893, 1897, 1903, 1912, 1913, and 1922.

Discharges and Gauge Heights.—The maximum discharge at Cairo for 1927, was 1 800 000 sec-ft., the actual height was 56.3 ft. on the gauge, and the confined height was estimated at 58.5 ft. The actual height at Vicksburg, Miss., in 1927, was 58.5 ft., which was 3.65 ft. above any previous flood, with a discharge of 2 278 000 sec-ft.—more than 20% greater than any previous flood.

Three waves passed Cairo in the last flood. The first was early in January with the Cumberland and Tennessee Rivers at high stage; the second early in February, with the Ohio River high; and the third in March and April, with the Upper Mississippi River high, and the Arkansas and White Rivers both at extraordinary stages.

In 1927, during maximum gauge at Cairo, the Ohio River contributed 814 000 sec-ft., whereas in 1913, it gave 1 395 000 sec-ft. One method of arriving at a future maximum flood has been to add the excess Ohio flow of 1913 to the Upper Mississippi flow of 1927. While this may be taken as the maximum possible, the maximum probable may be taken as somewhere between 2 150 000 and 2 250 000 sec-ft. at Cairo.

The average number of days of extreme flood heights is about 9 at St. Louis, 14 at Cairo, 13 at Memphis, Tenn., 19 near Arkansas City, Ark., 24 at Vicksburg, Miss., and about 25 at New Orleans, La. The number of days above ordinary high water is as much as five times this period.

The maximum discharge of the Missouri River is about 1 sec-ft. per sq. mile, that of the Ohio River is about 6½ sec-ft., and that of the Mississippi at Vicksburg, about 2.0 sec-ft. per sq. mile of water-shed.

The extreme range of gauge readings to date at Cairo and below, has been listed in Table 36.

The maximum flow at the latitude of Arkansas City in 1927 was 2 662 000 sec-ft., including overflow, and the actual discharge in the main river was 1 712 000 sec-ft. At Old River, the inflow was 2 010 000 sec-ft. and the outflow 1 461 000 sec-ft. in the main river, and 592 000 sec-ft. down the Atchafalaya River.

The 1927 flood was on the average 1.3 ft. higher from Cairo down, than the previous highest, or 1.7 ft. below levee grade. It exceeded the previous high gauge by 3.4 ft. at the mouth of White River, and was 2.2 ft. below the previous high at Cottonwood Point, Mo.

TABLE 36.—Range of Gauge Readings in Mississippi River.

Location.	Extreme low.	Extreme high.	Range, in feet.
Cairo, Ill.	−1.0	56.3	57.3
Memphis, Tenn.	−2 65	46.55	49.2
Arkansas City, Ark.	−3.6	60.55	64.15
Vicksburg, Miss.	− 6 5	58.5	65.0
Red River Landing, La.	−0.6	57.5	58.1
New Orleans (Carrollton), La.	−1.6	21.27	22.9

Since the Mississippi has no natural reservoirs, it is not surprising that there should be great variations in stage and flow, and that the river heights did exceed the previously expected maximums. The reservoir area of the Great Lakes gives the St. Lawrence a storage of more than 100 000 sq. miles. The Mississippi River has an average flow three times as large as the St. Lawrence, and a maximum flow nearly eight times as large.

The Levee System.—The present levee system had its beginning in the small mounds, about 2 ft. high, built at New Orleans as early as 1717. With each succeeding flood these small protections were built higher and took in more territory, so that gradually a levee system was created. This work was first done by private plantations, then by groups, and, finally, by levee districts and States. In 1859 the levee grade was only about 5 ft. above the banks. The United States as early as 1824 appropriated funds for river snagging, etc., and up to 1879, about $10 000 000 had been spent. It was not until 1879 that formal assistance on a large scale, both for levees and revetments, was given by the Federal Government.

At present, the levee system is about 1 815 miles long, of an average height of 17 ft., and about 472 000 000 cu. yd. in volume, which is about 260 000 cu. yd. per mile.

The expenditures to date (1927) have been as follows:

	Levees.	Revetment.	
United States.....	$71 000 000	$61 000 000	
Local co-operation	167 000 000	3 000 000	
Total	$238 000 000	$64 000 000	
Grand total			$302 000 000

The most recent authorization by the United States was in 1923, the project covering $60 000 000, of which about $30 000 000 had been spent at the beginning of the 1927 flood. The 1914 project was thus about 90% completed. The present local co-operation required by law is one-third of levee costs, and all right of way and maintenance. The United States constructs all revetment, except on tributaries, where local interests are required to co-operate.

The Mississippi River Commission.—This Commission was organized in 1879 to make complete surveys from the head-waters to the Gulf, and to prepare plans and execute works for navigation and protection against floods from Cairo to Head of Passes, La., including outlets, levees, closing chutes, bank revetment, and dredging. Jurisdiction was later extended to Rock Island, and to include tributaries in the flood back-water of the main river. The levee grade line established in 1914 was placed 3 ft. above the 1912 confined flood, the greatest to that time. The characteristics were: Free-board, 3 ft.; crown width, 8 ft.; river slope, 1 on 3; land slope, 1 on 3, to banquette, and 5 to 8 ft. below crown; and banquette, 20 to 40 ft. wide, with 1 on 10 slope and back slope 1 on 4.

Studies and plans are now being made to provide greater free-board, larger crown width, and flatter slopes. The new section would thus be much greater in volume for this average height.

Crevasses.—There were twelve principal localities where crevasses occurred. Counting the smaller breaks and those on tributaries, there were twenty-four all told under Commission plans. The first occurred at Dorena, Mo., 33 miles below Cairo, on April 16, 1927, and the last at McCrea, La., on the Atchafalaya River, on May 23. All breaks occurred on the right bank except two, one at Mound, Miss., on April 21, flooding Greenville, and the other at Caernarvon, La., on April 29, opened by the local interests to reduce the flood heights at New Orleans. There were heavy rains and high winds on April 20, with a rise of 1.1 ft., in 24 hours at Arkansas City. The Mound crevasse opened to ½ mile in 4 hours, and eventually flooded more than 700 000 acres, destroying many farm animals and flooding many towns. Mail placed in the street box at Greenville on April 24, was not delivered in Chicago for more than three weeks, showing the disruption of communication and transportation service. Four breaks occurred on the Arkansas River, and seven on the Bayou des Glaises and Atchafalaya Rivers. Many of the levees were not up to grade, nor under Commission jurisdiction. Some of them were undermined or cut by wave-wash and in spite of all the efforts of the local levee districts and

the United States District Engineers, they were overtopped or undermined and destroyed.

The flood has shown that protection must be given for greater floods, that levees in the geographical jurisdiction must be under Federal control, that the problem is National, and that participation by local interests must be made more liberal.

Caving Banks and Revetments.—The present average bank height is about 44 ft. above low water at Cairo, and about 45 ft. at Red River Landing, with an average slope down stream of 0.35 ft. per mile.

Caving banks are a notable characteristic of the lower river and can be prevented only by revetment. There are about 127 miles of revetment now built, consisting formerly of mattress and rip-rap, but more recently of concrete slab type. Several hundred miles of additional revetment will be needed. The cost is $300 000 to $400 000 per mile. The caving at times has equalled 1 000 000 cu. yd. per mile, in certain sections, the total amounting to 400 000 000 cu. yd. per year.

The lack of this revetment protection causes the need of expensive setbacks and relocations in the levee line, often costing more than $500 000 per year. At times, setbacks have a benefit of their own in providing more cross-section area for floods. Fixation of the bank line is an important part of regulation and flood control.

Suspended Matter (Sediment).—The amount of suspended matter present depends on the velocity, character of bed and banks, whirls, and eddies. It has been found to equal $\frac{1}{1\,500}$ of the discharge by weight and $\frac{1}{2\,000}$ by volume. There is also material rolled along the bed. The total quantity carried to the Gulf is estimated as equal to a volume of 1 sq. mile, 268 ft. high. The banks of the Southwest Pass have advanced at times at a rate of 1 mile in 20 years. The sediment deposited on the delta is estimated to average 0.01 in. in thickness annually.

Property Values and Damages.—The valuation of properties in the Delta Basin is estimated at $4 000 000 000 to $6 000 000 000; the damages of the 1927 flood at $200 000 000 to $400 000 000, of which $10 000 000 was physical injury to the railroads. The delta was overflowed over 50% of its area. More than 600 000 people were made homeless. The expense of the flood fight, for closing levee gaps and crevasses, and repairs, was more than $7 000 000. In addition, the levee districts spent more than $1 300 000 for the flood fight. The estimated cost of raising the wharves at New Orleans, and of right of way for necessary levee setbacks must also be considered. In addition, a thorough study of the economic considerations should be made at the earliest practicable date.

Future Protection.—Suggested remedies are control of drainage and contour plowing, reforestation, additional levee heights, dredging, reservoirs, outlets, diversions, and spillways.

Contour plowing and land drainage would give but meager help; they are impracticable because there is no law to enforce such a measure, and, as stated, the relief would be insignificant.

Reforestation should be done on its own merits. There is great need for such work. The benefit to flood control is problematic. An examination of the water-shed records in Minnesota shows that forests caused greater floods than cultivated land, but probably less than deforested scrub land or pasture. To include any forestation for flood control would require many years, even if the benefits were certain.

Any material raising of levee grades should be avoided, if possible. To hold in check the probable future flood would require a considerable raise unless auxiliary means can be used. These additional heights not only become very costly, but add a greater menace to the property and people behind the levees.

No particular change of the bed has been noted. If anything, there has been a lowering of 2 to 4 ft. between Cairo and Helena. The average depth of the thread of the stream at low water, taking both the pools and bars, is 31 ft. at Cairo, 48 ft. at Red River Landing, and 84 ft. at New Orleans. Dredging is necessary to maintain the present navigation project of 9 ft. by 250 ft. at all stages above Red River, and it costs about $600 000 per year. An enlarged channel width is advisable in view of the recent large increase in river traffic.

To dredge for flood control would require several hundred million cubic yards per year at an enormous cost. This would lower the bed only a few feet, and it would be filled again with the next flood.

Cut-offs would reduce the length of the river, but would increase the slope and velocity, and require new levees and revetments in the vicinity. On the whole, they would be a detriment to the river and are decidedly inadvisable.

At first thought, reservoirs seem to offer help. To be of real benefit, they should store water at least three months, from February to May; 1 000 000 acre-ft. of storage will hold back about 5 600 sec-ft. for 3 months, which is equivalent to about 1.07 in. of gauge on the main river. (About 60 000 sec-ft. corresponds to 1 ft. on the gauge.) Reservoirs are naturally divided into those above Cairo, those below Cairo, and those in the valley itself. To take care of 4 ft. of river height will require about 44 000 000 acre-ft. at a cost of more than $500 000 000. Additional storage becomes more expensive, since the most available and economical sites would be used first. The storage nearest the lower valley is the most advantageous, both in rainfall and in control. Storage for local flood control although simpler is nevertheless very expensive. The Dayton flood control, as an example, has cost $30 000 000.

In general, it is difficult and at times impractical, except at great cost, to use reservoirs as a direct means of flood control for the lower river. Where power is involved the benefits are necessarily divided, although still valuable especially to local flood control. Further study should be made of sites on the tributaries of the Lower Mississippi and of basin storage along the main valley below Cairo. Here, real benefits may be possible.

Outlets, diversions, and spillways seem the most promising remedies. A diversion channel from the main river near Arkansas City to the Red River is

possible, and an increased discharge is practicable through the Atchafalaya Outlet. Also, spillways are feasible above and below New Orleans. Any diversion above Arkansas City becomes much more expensive than the corresponding levee protection. The diversions mentioned will materially lower levee heights and will reduce the total cost over any plan without such diversions, in addition to giving greater security. It is reasonable and practicable that some solution based on revetment, on levees of proper section and free-board, and on diversion and outlets and possible reservoirs on the lower tributaries will secure the necessary protection.

Conclusions.—Complete protection against floods such as that of 1927, and of future probable floods, is vital to the security and prosperity of the Mississippi Valley. The business and transportation interests, not only of the valley, but of the nation itself, cannot be harassed by recurring floods. The cost of the protection may run into hundreds of millions of dollars. The economic conditions should be studied, and a comprehensive plan submitted, based not only on sound engineering, but on considerations of National obligation and of lasting benefit to the whole country.

Q. C. Ayres,* M. Am. Soc. C. E. (by letter).—Like many another engineer whose livelihood does not depend, directly or indirectly, upon the methods used in the control of the Mississippi River, but whose inclinations lie within the general field of hydrology and hydraulic engineering, the writer has read with intense interest the very valuable and illuminating papers contributed to the Flood Symposium. He recalls the many perplexing questions that assailed his mind in 1913, when he took part in the memorable high-water fight of that year at Greenville, Miss., and he notes that most of these questions still remain unanswered. Also, the many months of strenuous activity in connection with swamp surveys in the shadow of the mighty river are not forgotten.

An outstanding feature of this Symposium seems to be a sharp difference of opinion, accentuated to an extent not commensurate with the large amount of attention devoted to this subject in the past. A moderate degree of such variance is, of course, a normal, stimulating, and desirable thing, but it would seem that radical differences would have been ironed out before now. If the recognized leaders of the profession, both within and without the Mississippi River Commission, can not agree among themselves on fundamental conceptions of river control, how can they expect anything but exaggerated disagreement among the rank and file? And what can that breed but still more accentuated misunderstanding in the mind of the voting public? Since the Congressional mind is nothing more or less than a composite cross-section of public beliefs, inflamed by diverse political and partisan interests, it would seem that flood legislation in the past has been all that could have been reasonably expected. After all, under the American system of government, no matter how competent and ingenious an engineering body may become, any program that might be devised must be patterned to fit the specifications of purse-holding laymen.

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It is the writer's belief that, when all the facts are considered, the record of the Mississippi River Commission in the judicious handling of a very difficult and many-sided problem will measure well up to the standards of contemporary engineering practice.

The writer particularly has in mind Mr. Morgan's paper. The case he makes against the Commission seems just a little bit too plausible. For example, he quotes the President of the Commission as having stated in 1912 that $73 000 000 was ample to do all the remaining necessary work on the levee system. This is no doubt a correct statement of majority opinion, but the writer happens to recall that the individual estimate of a civilian member of the Commission—C. H. West, M. Am. Soc. C. E.—at that time was $250 000 000.

This circumstance would seem to blunt the point of Mr. Morgan's sweeping denunciation on one score at least and leads to the belief that perhaps other scores would be similarly relieved of harshness if all the pertinent details were known. The impression of blind "ancestor worship" so artfully created by Mr. Morgan as the outstanding trait of the Commission personnel seems subject to discount on its face. Furthermore, it is an anachronism to state that the Commission failed to apply 1927 methods to the problems of the preceding century.

Happily, engineers are agreed upon one matter of transcending importance —the need for accurate and comprehensive data. It would seem almost self-evident that the first step in the solution of any scientific problem is the collection of adequate and indisputable facts; yet this basic phase of the Mississippi question has been undeniably neglected to a large extent, under pressure of restrictive legislation and limited funds. Lack of conclusive information doubtless explains in large measure the divergence of opinion so regrettably apparent, but for this, any indictment of the Mississippi River Commission must be leveled also at the entire profession.

There is reason to hope, however, that Congress will enact new legislation substantially in accord with the latest recommendations of the many agencies now at work on the problem. Facilities may thus be provided for the first time to gather a great mass of physical data, to enlarge the vision and diversify the engineering talent at the disposal of the Commission, and to enable many important controversial theories to be definitely proved or disproved. In such an event, the present moment may mark the end of a long period of pioneering with expedient disbursements, and the beginning of a new epoch more in keeping with the ideals of the profession.

H. S. Gladfelter,* M. Am. Soc. C. E. (by letter).—The paper by Major Godfrey is of much interest in presenting briefly, the extent of the development of modern transportation on the Mississippi River and the extent to which flood protection measures may be aids to navigation.

With the gradual building up of rail-transportation systems, running parallel to the Mississippi River, old-time steamboats gradually disappeared. These old steamboats had no regular schedules, and their rates were whatever

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the traffic would bear. They offered no insurance that freight entrusted to them would ever reach its destination. No adequate terminals were provided for the exchange or storage of freight after it was delivered. The designs of boats and their mechanical equipment did not change materially in the period from 1850 to 1900. In 1852, when steamboating was at its height, 3 100 boats docked in the Port of St. Louis, and the average load of these boats was only 26 tons.

Contrast this condition with that now existing. The Federal Barge Line operates 13 powerful boats that handle tows of 8 000 to 12 000 tons. These vessels are equipped with modern propellers and triple-expansion condensing engines. The American Barge Line (referred to as the W. C. Kelly Barge Line) with four high-powered Diesel towboats, handles tows of from 3 000 to 4 000 tons, and the giant stern-wheel towboat, *Sprague,* with modern machinery, having a power plant of exceptionally high efficiency, handles tows consisting of 224 000 bbl. of crude oil, or about 40 000 tons.

Major Godfrey refers to experiments now being conducted with pulverized coal as fuel on one of the boats of the Federal Barge Line. If these tests are successful a new field of cheap fuel will be available for use on the larger vessels. With high labor costs, it is essential, in the operation of river boats, that machinery of the most efficient type be adopted. The old days of inefficient machinery are gone.

The use of modern terminals with machinery for handling the freight is emphasized. Modern terminals have been erected in a number of cities from St. Paul, Minn., to New Orleans, La. These have car interchange facilities. In some cities even store-door delivery is attempted by motor truck, in order to compete with rail transportation. River transportation, to be successful, must give all that rail transportation offers and more.

Considerable interest has been taken in the statement in the message of President Coolidge to Congress that the Federal Barge Line, having demonstrated that it can make money, should be turned over to private interests as soon as possible. With the competition offered the railroads by water transportation interests, the question naturally arises, as to whether the railroad companies will be allowed to acquire control of the water transportation and, in order to protect their rail interests, gradually throttle the water transportation as was done in the past.

The earliest system of aids to navigation consisted of regulation works in the form of permeable dikes. These seem to have been effective for a short time in improving channel conditions, but due generally to insufficient funds, were not properly maintained, and, eventually, all the dikes failed. This system of regulation, if applied to the entire length of the river and properly maintained, would probably have resulted in good channel conditions, but the expense of maintaining a channel in this way would be very much greater than the cost of maintaining the dredged channel, adopted later.

The assistance offered to navigation by dredging, channel-marking, and channel charts, has been effective in providing a channel suitable for present requirements. Under this system, the commerce on the river has increased

rapidly. The constantly increasing volume of navigation and the economic size of tows will undoubtedly, in the future, demand a less restricted channel and one of greater depth and width. Dredging alone will not be effective in producing the required channel economically.

The plan of flood control of the Mississippi River presented by the Chief of Engineers to Congress recognizes the limitations of dredged channels and provides for a system of regulating works similar to that used successfully on the Mississippi River between St. Louis, and Cairo, Ill. This system of regulation would serve the dual purpose of bank protection and channel improvement.

With the work embraced in this flood-control plan, navigation interests are assured that channel conditions will be improved; and while flood control and improvement of navigation are entirely different problems, it will be found that many of the solutions of flood control will also be solutions for providing improved navigation.

Byron E. White,* M. Am. Soc. C. E. (by letter).—The excellent and authoritative papers presented on this subject have done much toward clarifying the atmosphere of uncertainty which has shrouded the Mississippi River problem. It is fortunate that it has been so fully treated, in its various phases, by eminently qualified engineers.

A careful reading of all the literature bearing on the question, leads to the conclusion that there is no escape from the construction of levees along the lower river, as the main line of defense against floods. It is also quite clear that the raising of levees to greater heights cannot go on indefinitely. Some auxiliary means, therefore, must be adopted to care for the excess flood waters, beyond the safe capacity of the channel formed between the levees.

The only means which has been more or less generally accepted as practicable are spillways and auxiliary flood channels. This is the plan so well described and advocated by Mr. Garsaud. The greatest objection raised to spillways is the formation of bars or shoals down stream therefrom in the main river channel and the resulting reduction in carrying capacity, etc. If such spillways are controlled, so that the flow in the river down stream may be kept at a safe height below the tops of the levees, and only the excess water diverted, such shoaling should be effectually prevented. Colonel Kutz states his belief that such will be the case.

Such spillways or sluice-ways (as they may properly be called) will be of unprecedented dimensions and capacity, but, otherwise, not unlike hundreds of similar structures. They should present no unusual difficulties in construction or operation. Their proper use should be of great assistance in maintaining the main river channel during and after floods and minimizing the work necessary to maintain navigation channels after a flood has subsided.

Any doubts as to the proper arrangement, location, and performance of such controlled spillways could be rather quickly, cheaply, and easily proved or dispelled by carefully conducted tests of models in a hydraulic laboratory, such as has been so strongly urged by John R. Freeman, Past-President,

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Am. Soc. C. E. Many other detail problems of levee construction, control of erosion, and channel control could be studied in such a laboratory. Such immense sums of money spent in construction, and such vast interests, both financial and human, are involved in the success or failure of the plans and structures necessary to control the destructiveness of floods, that the relatively small cost of model tests fades into insignificance when the certainty of results and other advantages are weighed against it. The study of these problems in a laboratory should do much to popularize model testing among engineers and executives and aid immensely in putting American design and construction work on a more sound and satisfactory basis, especially where novel designs are involved.

While this may cause some delay in the preparation of final plans and the execution of the work, the certainty of results should more than compensate for it, especially if the existing levees and other structures are promptly repaired and made ready for service before the next flood season. It would be almost criminally wasteful to embark on a radical change in plan, without first making sure of its efficacy in detail. Unless there is a real threat to the stability of the levees, the writer is also strongly in favor of the construction of wide, paved highways along them, for ease of access, inspection, and repair.

Much doubt as to the feasibility of reservoir control as a means of preventing, or even materially reducing the height of floods in the lower valley is expressed by practically all who are familiar with the river. The physical dimensions of a dam which would impound the requisite volume of water might be very great, without exceeding the value of a large regulating capacity. Hence, it is just possible, even if a little improbable, that, on account of the great length of dam which would be involved, some feasible site may have been overlooked in all the investigations made thus far. Another search for such sites may well be in order, before definitely abandoning all idea of partial control by means of reservoirs.

In view of the assertion that only a relatively small part of the lower valley has been topographically mapped, it is not difficult to conceive that some, perhaps several, feasible dam and reservoir sites may have escaped proper attention. A survey by the rapid methods of the U. S. Geological Survey, covering those areas which hold forth any promise, would give quick results and disclose any sites, worthy of further and more detailed investigation. It is, of course, possible that such studies have been made and that there are no potential dam sites which are not known and have not been studied, but the discussion has not shown this to be the case.

The almost infinitesimal effect of reservoirs located on the Upper Mississippi and Ohio Rivers on flood heights at Cairo, Ill., cited by Colonel Kelly, may seem astonishingly small, yet they are probably not far from the truth. Therefore, any effective reservoir control must be located (as stated by some of the authors) much closer to the region to be protected and must store vast quantities of water and cover immense areas of land. Whether such areas exist, and whether they are of small enough property value to warrant their use for this purpose, does not appear to have been authoritatively stated.

The study of reservoir possibilities referred to by Colonel Kelly does not offer much encouragement for the hope that economical reservoir sites, effective for flood control, may exist in or near the lower valley. It is, of course, possible that in the haste with which the investigations and report had to be made, something of value has escaped attention or has not been properly investigated or evaluated. The case should not be considered closed until every possibility has been completely studied and its value deliberately weighed.

Reforestation has also been frequently advocated as a means of reducing the height of floods on the Lower Mississippi and elsewhere. It is generally recognized that forest cover is quite effective in limiting or preventing soil erosion, as well as helping to retain the water absorbed by the soil. Record floods, such as that of 1927, are the result of prolonged, intense rainfalls, and often (although probably not in this case) the rapidity of run-off is augmented by frozen ground. Under these conditions the effect of forest cover on holding back the water must be very small, especially when the ground is already thoroughly saturated, as was the case in 1927.

The work, time, and expense involved in reforesting, say, 20% of the total drainage area (1 250 000 sq. miles) of the Mississippi, or 250 000 sq. miles, are enormous. At the rate of 1 200 trees per acre, there would be required 192 000 000 000 trees and the cost would be from $1 500 000 000 to $2 000 000 000.

It is hardly possible that a much greater area, aside from that already forested, could be devoted to that purpose, without encroaching on lands which are more suitable for agriculture and grazing. When the percentage of the total area available for reforestation is taken into account, together with its inability to greatly limit run-off under exceptional conditions, this does not appear to offer much encouragement as a means of flood control. Since the effect of such reforestation as may be practicable will not be great, it would be advisable to consider it as another line of defense against the possible maximum flood.

These remarks must not be misconstrued as opposing reforestation. The immense value of an adequate timber supply, about 30 to 50 years hence, is of itself sufficient reason for encouraging the reforestation of all areas where trees can be grown, as a crop, to better advantage than anything else. The comparatively small benefit of forests as a means of flood prevention, the percentage of the drainage area available for forest planting, and the length of time and the difficulty of getting an effective reforestation program started and carried to completion, all rule it out as an immediate or major factor in this situation.

The events of 1927 emphasize the desirability and, indeed, the necessity for placing the planning and supervision of the construction of all works for the control of the Lower Mississippi in the hands of a single agency of the United States Government. Further, in order to stem the rising tide of paternalism and the reliance on the Federal Government in all kinds of major projects involving local benefits, the States, cities, and business interests should be required to shoulder a reasonable portion of the costs. To do other-

wise will constitute a precedent which will be used as an argument in unnumbered future (as well as present) projects. Once the principle of local sharing of costs is abandoned, there will be no end to the calls for contributions from the Federal Treasury to do that which is the duty of individuals, communities, and States. While the nation should help, where navigation and national interests are involved, its generosity should not be imposed upon. The loaning of the credit of the nation in an emergency is a different matter.

In regard to Mr. Scheidenhelm's excellent paper, on "Power as Affecting Flood Control", there are a number of points which might well be amplified.

In the face of the already low and constantly declining cost of fuel-generated energy, the value of stored water for power generation is so small that, unless the cost per unit of volume is exceptionally low, storage reservoirs, exclusively for that purpose, are not economically feasible; that is, unless there exists a large aggregated head down stream from them, which it is economical to develop for power production. In the usual case, this means that, unless the available head, or fall, below the reservoir site amounts to several hundred feet, the cost of the reservoir may be prohibitive. This fact eliminates many apparently promising streams from consideration as power producers. As steam power costs go lower (and further economies are constantly being effected), this effect will be greater and will continue to narrow the choice of streams on which economical hydro-electric plants may be constructed.

In the many cases where suitable reservoir sites are found above large available heads, the head at the reservoir dam itself will usually be found to be a small part of the total involved. If generating equipment is connected to the outlet conduits, in many cases, it will be a producer of secondary energy, only, as and when water and a suitable head are available. When such an installation is a part of a large interconnected system, this arrangement is usually entirely feasible and practical, provided the costs are not too great.

In favorable locations, an additional head, immediately down stream from the storage dam, may often be combined with that at the dam by extending the outlet conduit down stream. When this additional head is equal to, or in excess of, that at the dam, power and energy, with properly designed equipment, may be generated from any water released, down to the practical drawdown level in the reservoir.

In this as well as in the other case, previously described, a firm value can only be attached to such power and the corresponding energy, as is produced from the minimum quantity of water released from the reservoir, either continuously, or at all times when firm or dependable capacity is required. If, at any time when firm power is needed, water cannot be released, or the head is so low that no power may be produced, then the connected power plant will have no firm capacity value. The value of such a plant when incapable of producing firm power and energy is much reduced, because it must be based on the value of secondary, or intermittent, energy only. Where steam or fuel-generating plants are connected to the system, the upper limit of this value is the cost of fuel per kilowatt-hour, plus the additional labor and other charges caused by operating the steam plant, less the cost of trans-

mission to the market which would otherwise be supplied by steam power or other alternative source. In many cases there are other conditions or costs which still further reduce the value of the available power and energy.

In the case of reservoirs, where such additional head is not available, and it is desired to draw the water as low as practicable, the installation of a power plant is only justified when the intermittent supply of energy has a value sufficient to pay the charges on the plant, plus a reasonable profit. In such cases, therefore, the installation and operating costs must usually be exceedingly low, if the plant is to be justified. Herein lies the best reason why power plants are not usually found at the foot of storage dams.

Where storage for flood control and storage for power are sought to be combined in a single reservoir, the flood-control requirement will usually intensify the conditions unfavorable to power generation at the foot of the dam; and where additional head is available immediately below, it will, of course, operate to reduce the power and energy output, and, consequently, the value of a connected hydro-electric plant. Also, where both flood control and storage for power are combined in the same reservoir it is sometimes, although not frequently, possible to increase the height of the dam and utilize the upper portion, where the proportion of volume to depth is greatest, for flood control. If, in this, as in the preceding case, the contemplated total drawdown of the reservoir is too great a part of the total head available, the head will finally become so small that no power can be generated. The value as a power site is then greatly reduced, if not altogether wiped out.

Many of the reservoirs at the head-waters of the Mississippi, the Ohio, and other tributaries of the Mississippi, will be beneficial to power development, especially those in the mountainous regions. They will also exert some slight effect on floor reduction in the lower valley, as described by Colonel Kelly. Those lower reservoir sites, which would exert the greatest influence on flood control along the lower river, will be practically, if not entirely, useless for the production of power. It is unlikely that basins will be found at proper locations, the capacity of which will be sufficient to accomplish completely the desired purpose, even if designed as retarding reservoirs, like the Miami Conservancy District structures. It will be essential, therefore, to empty them as rapidly as possible, after each filling, in readiness for the next flood. The head, therefore, will be extremely variable, and power will be available only at long intervals, and for short periods of time. Hence, there is slight possibility that power will be a by-product of their construction.

Favorable effects of reservoirs on flood heights are, of course, most noticeable immediately below the dam, and it is possible that the sequence of arrival of flood crests from different tributaries at some point down stream, as, for instance, at Cairo, may be such that the flattening and lengthening of the flood peak above the crest of a storage dam may have unfavorable results far down stream. Hence, reservoir control at distant points does not have the beneficial effect that the same capacity would have, if concentrated near the point where its effects are most desired.

The recent development of large interconnected power systems has made it feasible and economical in many cases to utilize the intermittent power and energy available at the foot of reservoir dams, when conditions for them are favorable, and when within a reasonable distance of existing transmission systems and markets. Interconnection on a large scale has also made it economically possible to combine storage for flood control and for power in many instances, where it would otherwise be uneconomical for either purpose alone. Where such cases exist, the just allocation of costs between these uses is a matter of great importance in order that neither interest may be unjustly treated or taxed. The proportion of the cost which power interests can afford to pay is strictly limited by economic conditions. Expert engineering, legal, and business advice, as well as a sense of fairness and open-mindedness, are, therefore, essential in dealing with such a problem. With such an approach, a reasonably harmonious compromise of the conflicting interests of storage for power and flood control, may often be reached, to the benefit of all interested parties.

The degree of conflict between the requirements of storage for power and for flood control, are proportionate to the relative extent of stream-flow regulation which may be effected by the reservoir or system of reservoirs in question. If complete, or nearly complete, regulation may be attained, the interests need not seriously clash. It then becomes merely a matter of draining the reservoirs well down in advance of the usual flood seasons. From this case to that of the relatively small reservoir effecting little regulation, the interests become more diverse. Nevertheless, large benefits may usually be obtained by both interests, by the exercise of fairness and common sense. Here, the connection of the power plant or plants to a large interconnecting system may likely be the saving grace which makes the generation of power at all practical and economical.

In general, the writer believes that while the requirements of power and flood control in a reservoir are more or less at variance, unless the necessities of one or both are decidedly fixed, both may be reasonably well accommodated, to the economic benefit of the community and the interests involved. A factor of equal and prime importance, which seems to be often overlooked or forgotten in some current discussions, is the existence of a satisfactory market, capable of absorbing the output in a reasonable time after the supply becomes available. If the anticipated absorption runs into too long a period of time, the interest and carrying charges on the unutilized portion of the investment, which must be added to the capital account, may make the ultimate cost altogether too high. If an adequate and fairly immediate market is not assured beyond a reasonable doubt, there is no justification for the construction of the project; and again, beyond a doubt, the output must compete in cost and dependability, with the other available sources of supply.

In the pressure for reservoir construction, it is often assumed that hydroelectric power supplies will be created which will assume a large proportion or all of the costs. It is sometimes asserted, also, that desirable reservoir or power sites are being allowed to lie dormant for ulterior reasons. It is well

to bear in mind at all times that, with few exceptions, every existing market is adequately supplied, and will continue to be well supplied, up to the moment when the output of a new development becomes available, barring unexpected and unusually large demands. The ancient law of supply and demand, coupled with good business and engineering judgment, may be depended upon to bring about the development of the most economical potential source of available supply.

There are projects of such magnitude, or remoteness from any adequate present market, that any corporation or other authority may well hesitate to embark on their immediate development and the assumption of the annually mounting carrying charges which must accrue before their entire output is successfully marketed. These facts and factors, affecting each project, must be carefully and conscientiously weighed before arriving at a final decision, or the results may be unhappy, if not disastrous.

W. G. PRICE,* M. AM. SOC. C. E. (by letter).—The Mississippi is a river burdened with clay and sand that has flowed in from its tributaries during the ages, to build land in an arm of the Gulf of Mexico.

Until Man began building levees much of the clay and sand was annually spread over the land which the river had built, and which was thus being continually raised. The river developed a channel sufficient to carry only its normal flow of water. Then Man built levees to stop the flood water from spreading over and further building up the land; and these levees, in order to be efficient, had to be high enough to carry all the water of the greatest floods, which is impossible. The engineers in charge reasoned that when the river was confined between levees the increased power of the resulting swifter current would erode and enlarge the channel and increase the carrying capacity so that the height of the levees need not be extended. This reasoning has been proved false.

In 1879 and 1880, during the low-water season, the writer measured, for the Mississippi River Commission, the sand waves in the river at Plum Point and Fulton, Tenn. Many of these waves, which closely followed each other at Plum Point (where the river was very wide and shallow) and at Fulton (where the river was narrower and deep) were as high as 10 ft., with a long slope on the up-stream side, and they moved toward the Gulf many feet each day.

A large quantity of sand was repeatedly caught in a trap bucket more than 5 ft. above the bottom of the river. The writer also measured the large quantity of silt and sand caught at the surface in the almost continuous boils (which are caused by the river running over the sand waves) in the Lower Ohio and Mississippi Rivers.

At Fulton, where the river was very deep, the many streaks of boiling water, spaced apart (which indicated the location of the crests of the sand waves and their stream lengths), could be seen extending nearly across the river. These data indicated that the scouring and material-carrying power of the Mississippi is very great at low water and immensely greater during floods when the river is confined between levees.

* Cons. Engr., Yakima, Wash. Mr. Price died on July 6, 1928.

Cross-sections of the Mississippi were made at many places in 1879 and 1880, and these, when compared with sections made since, indicate that the river bed is neither rising nor falling. These data also indicate that the quantity of heavy material being carried in by the tributaries and the quantity supplied by the caving banks are equal to that being dredged and carried out to the Gulf.

The river, by wandering about and digging away its banks as well as some of its levees, is now carrying to the Gulf an immense amount of material which it deposited and stored during past ages. The data also indicate that, if the river were stopped from wandering about, and thus scouring material from its banks, the present rate of erosion along the bottom would soon lower its channel and the heights of its floods.

There are only four ways that can be used to control floods in the Mississippi; these are, by the construction of levees, by-passes, spillways (also called outlets), and bank protection to cause the river to dig its bed deeper. Along the river from Cairo, Ill., to the mouth of the Arkansas, levees have failed and they will not give safe protection if built higher. The land has a downward slope away from the river and the flood currents are continually eroding the bank and following down this slope and, therefore, the new levees must be higher.

In this reach neither by-passes nor spillways should be located, therefore, the engineer is limited to the use of bank-protection works. Since 1883 much bank revetment has been constructed by the engineers in charge of this work and it has been destroyed wherever the swift current ran against it.

Practically all these bank-protection works have been of one type, which indicates that the engineers have been unable during forty-four years to produce new or better plans, but have continued to spend appropriations for the construction of worthless designs. The writer's experience in building a different type of durable revetment in the Mississippi indicates that works can be built to protect the present banks and also those against which the swift current from a cut-off would run.

There are several loops in the river where cut-offs can be made, thus creating an increase in the velocity and power to dig the bed deeper.

From the mouth of the Arkansas to the mouth of the Red River, a by-pass through the Tensas would carry enough of a great flood so that the levees along this reach of the Mississippi need not be excessively high and bank-protection works would not be needed, except to protect property and to avoid the continual expense of building new levees. This by-pass should take only the quantity of water required to keep the flood height safely below the top of the levees in the Mississippi; otherwise, it would defeat its object by causing the cross-section of the river to become smaller. It will really be an outlet since its waters will not return to the Mississippi. The by-pass will not close itself. It will form the size of its channel to correspond with the amount of flood water it has to carry, which is the law of all streams in material that is easily eroded.

If the Mississippi were permitted to choose its own route from the mouth of the Red River to the Gulf, it would go by way of the Atchafalaya; but for

commercial reasons and to avoid the flooding of valuable plantations along the latter river, this outlet must continue to take only the water that its channel can safely carry with its levees remaining at the present height.

No spillway should be permitted to take water from the Mississippi between the Red River and the Gulf when the water is below the safe flood height. If the spillway is permitted to flow when the river is lower, the section of the channel and the discharge capacity of the river below the spillway will be reduced by a fill and the spillway will not reduce future floods.

Sufficient water should be permitted to flow to the Gulf by way of the Atchafalaya or Vermilion Bay spillways, or by both, as proposed by Mr. J. P. Kemper,* to render spillways into Lake Pontchartrain unnecessary. A wide shallow spillway should be provided below New Orleans, La. This spillway should not lower the flood in the Mississippi enough to cause a fill in the river below the spillway; otherwise, the flood-lowering effect of the spillway at New Orleans would be eliminated. A river, in an alluvial land automatically digs or fills its channel to a size proportional to the quantity of water ordinarily flowing through it, and it will dig somewhat deeper if its outside curved banks are prevented from caving.

The writer is not familiar with the land through which Mr. Kemper's proposed Vermilion Bay outlet would flow. If it is to be opened, work on it should begin without delay to avoid another disastrous flood in Lower Louisiana. The Tensas and Poydras spillways can be opened quickly when another great flood is approaching.

In prehistoric times, the Red River was not connected with the Mississippi. It flowed to the Gulf by way of what is now the Atchafalaya. Great dams, or what were called rafts by steamboat men, were formed in the Red River and one in the Atchafalaya, above Melville, La. About 1875, the State of Louisiana removed these rafts, but did not make a good job of it, because later two steam tug propellers were broken while trying to pass through the Atchafalaya raft. These rafts were formed of great trees which had caved into the river. Their roots were buried in the bottom of the river and their upper ends were spears, pointing down stream and thus menacing the approach of steamboats. Forty years ago (1888) the remains of these rafts in sharp bends in the Red River, the writer noticed, were efficient bank revetments. The Red River, in prehistoric times, having clogged itself by a raft in the Atchafalaya, cut through into the Mississippi and, later, the Mississippi made a cut-off and thus formed Turnbull Island.

The combined outlet of the Atchafalaya and Vermilion must carry the excess of bank-full stage that can safely be permitted in the Mississippi. The Vermilion outlet should have a wide, shallow opening at its upper end so that it cannot take water from the Mississippi below that level, except that it should have a narrow canal deep enough for navigation. This outlet has a slope much greater than that of the Mississippi and, if not controlled, will become the great river, while the Mississippi with its lesser slope, will become a bayou, with a sand-bar filling its channel near Red River Landing.

* In a report to the National Flood Commission, entitled "Plan for Control in the Mississippi Valley," by J. P. Kemper, August, 1927, New Orleans, La.

OREN REED,* ASSOC. M. AM. SOC. C. E. (by letter).—This Symposium is the most thorough discussion of the Mississippi River flood control problem that has been published in recent years. The papers were prepared by engineers who have given many years of labor and thought to the control of a mighty river. An exceptional flood has given a full test to the measures of control that have been attempted.

The people of a vast fertile area are asking for relief and protection from flood damage. The country as a whole has at last realized the magnitude of the problem and is in the mood to demand the application of effective remedies for the control of floods in the Mississippi area. It is, in a measure, unfortunate that the engineering features of the problem could not have been more fully decided before Congressional action was started. It is generally admitted that the Mississippi River Commission has made good use of the funds available. Experience has shown that the use of levees has been well justified. However, for physical and economic reasons, levees probably should not be trusted to carry the maximum possible flood which might occur below Cairo, Ill. Reservoirs, spillways, and by-passes should all be studied and used where feasible.

Shortly after the Mississippi flood of 1927, a part of the Duchy of Liechtenstein was covered by the flood waters of the Upper Rhine River. Certain problems developed which were similar to those presented by the Mississippi flood. This Duchy is in the northeast foot-hills of the Alps, and the river at this point has a moderate slope and is bordered by a flood-plain of considerable width.

Levee construction was started about 100 years ago. Since the channel has been fixed between two parallel levees, the stream bed has risen because of the flat gradient and heavy load of débris carried by the stream. In places, the present stream bed is 23 to 29.5 ft. above the land on both sides. At Buchs, where the first crevasse occurred, the stream bed has been raised 11.4 ft. since 1848. Corrective work had been done in recent years in an endeavor to stabilize the channel at its present elevation.

Below Liechtenstein, the Rhine flows into Lake Constance. Above the lake there are two levees on both sides. The low-water channel is confined by a low levee, and the flood levee is set back far enough to give ample floodway. This system of levees is favored because the narrow low-water channel prevents the dropping of the silt and débris load.

The growing danger at Buchs due to the raising of the channel bed was fully understood. The levee on the Swiss side had been raised 1 m. (3.28 ft.) in 1926, and work had been started on the Liechtenstein shore. Two bridges crossed the Rhine at Buchs, a wooden highway bridge and a steel trussed railway bridge. These bridges had not been raised at the time of the flood.

The head-waters of the river were swollen by a severe two-day storm in the last days of September, 1927. At some places, precipitation equalled nearly 7 in. in 2 days. This is, however, not the greatest precipitation on record in Eastern Switzerland. The 2-day precipitation in 1868 at Barhardin

* Asst. Designing Engr., San Joaquin Light & Power Corporation, Fresno, Calif.

Pass was 18.4 in. In 1910, the rainfall was also more intense than in 1927 and covered a greater area.

The quantity of water carried by the Rhine above Lake Constance on September 25 and 26, 1927, was the greatest yet observed. At Buchs, the flood became dammed by the bridges, broke down the weaker highway bridge, and overflowed the Liechtenstein levee above the railroad bridge. When the crevasse occurred, the Swiss levee still had a free-board of 1.3 ft. The break widened quickly, so that a large part of the flood water flowed into the adjacent lowlands. After the high stage was passed, the full flow of the Rhine ran unchecked into the flood-plain, leaving the old river bed dry. The flood waters ran back into the Rhine near the mouth of the Ill River about 10 miles below the break at Buchs. The land side of the levee was washed by the flood waters, thereby causing three secondary breaks.

The influence of the flood on the level of Lake Constance produced no serious results. If a part of the flood had not been held back in the Rhine flood-plain, the level of Lake Constance would have been raised about 10 ft. The maximum discharge from Lake Constance was about 141 000 cu. ft. per sec., which was slightly less than in 1910.

It is planned to organize an international commission for the control of the Rhine from the source to Lake Constance and transfer all fees and facilities for river control to this body. A comprehensive plan can then be formulated, which will not be dependent on political boundaries. It is proposed to regulate the mountain torrents so that a large part of the débris will be retained by check dams.

HAROLD C. FISKE,* M. AM. SOC. C. E. (by letter).—Reservoirs have been created to store water for power purposes and they have been proposed and built for the storage of surplus water at times of flood. It now seems to be taken for granted by many that these two uses are necessarily antagonistic, and that if the reservoir is operated successfully for one purpose it will be a failure for the other. Furthermore, it is frequently concluded that the flood reservoir must be totally emptied immediately after local danger has passed, by discharging at the maximum rate which the channel below will carry. The large volume of water thus sent down stream will meet a flood wave in some section below and will increase instead of diminish the flood risks at and below that point.

Mr. Scheidenhelm has done a very valuable service in bringing up for further and more careful consideration the assumptions on which these conclusions are based. Very properly, he points out that Nature has given a perfect example of exactly the opposite argument in the Great Lakes, where the requirements of both flood prevention and power development are met to an ideal degree. Of course, this well-known example is commonly dismissed at once by comparing the size of the Great Lakes with that of any artificial reservoir that has been or could be constructed by Man, with the value of the territory that would have to be submerged to duplicate the lakes, etc.

What Nature seems to be suggesting is the value of studying her methods and of determining how far Man can go in any given case, not necessarily

* Lt. Col., Corps of Engrs., U. S. A., Fort Lawton, Seattle, Wash.

in duplicating her dimensions, but in securing her results. Since Man cannot hope to match her in all respects he seems sometimes to forget that flood damage may, in general, be prevented by the storage of a part of the excess flow. He forgets, also, that there is some chance to release this storage, not blindly the moment local conditions permit, but after a longer interval. It is clearly impossible to state a fixed rule that will solve this problem to the satisfaction of both flood and power interests under all conceivable conditions, but if each particular case is studied carefully on its individual merits it would seem reasonable to hope that in a certain percentage of instances a solution would be found that was satisfactory to all concerned.

As an example may be cited certain features of the survey of the Tennessee River and its tributaries, authorized by Congress in 1922 and on which a final report is expected early in 1928.* This survey is to cover the water resources of the Tennessee Basin and to include full consideration of the needs of navigation, power, flood control, and other allied topics.

One of the many different sections to be studied was the basin of a large tributary, the Clinch River. On this, prior to the survey, private power interests had in mind the construction of a dam 175 ft. high which would create the largest feasible reservoir for power purposes. The correct installation was found to be about 160 000 h.p. In studying this section the survey, in addition to power, gave careful consideration to the needs of flood control, navigation, etc. It may also be stated that due attention was given to all items of cost and to all sources of profit.

It was found that below this storage dam six other run-of-river plants were indicated in the next 250 miles, three on the Clinch River and three on the Tennessee River above the existing Hales Bar Dam. By diminishing or shutting off the flow from the Clinch storage during flood seasons the tendency to drown out the three Tennessee plants was reduced and their efficiency at such times was increased. By including in one unit the Clinch storage, the Clinch run-of-river plants, the three run-of-river plants above Hales Bar, the Hales Bar plant, and the plants at Muscle Shoals, there was obtained a development of tremendous size. Its value was greatly augmented by the volume which could be released at low water from the Clinch storage. At flood seasons discharge from the Clinch storage could be diminished or stopped, leaving only the Tennessee River plants in operation. Since the Tennessee would then be at a high stage there would still be ample flow for these plants to operate at full capacity, and the result was substantially the same as that obtained at low water with the entire system operating under the increased flow from the Clinch storage.

As for the flood situation, there were available the daily flow records for a period of twenty years, together with incomplete records going still farther back. No attempt was made to discuss or solve hypothetical conditions, but for each day of this 20-year period the output of power and the daily height of the water in the reservoir were determined. This period included one disastrous flood and several that did material damage at Chattanooga, Tenn., about 250 miles below the storage. For one short period, water flowed over the crest of the dam but, as far as floods are concerned, the quantity was

* This report was presented to Congress in December, 1927.

negligible. For the remainder of the time the discharge was controlled, and at all times the Clinch River was eliminated as a material factor in Tennessee floods. The height of the storage dam was increased from 175 to 225 ft.

To show the change in technical and general public opinion in this connection a few quotations from official documents may be convincing. House of Representatives Document No. 319* sets forth conditions as they were known before the survey was authorized. House of Representatives Document No. 463† contains a partial report on the survey and outlines some of the information obtained to that time. Referring to the Clinch storage, Document No. 319 gives the following (in terms of installed capacity):

Dam No. 3 (storage)	160 000 h.p.
Dam No. 21	80 000 "
Total	240 000 h.p.

In reference to the same section, Document No. 463 gives the following (also, in terms of installed capacity):

Cove Creek (storage)	200 000 h.p.
Clinton	15 000 "
Melton Hill	60 000 "
Kingston	50 000 "
Total	325 000 h.p.

From this it will be noted that, on this section of the Clinch River, combined navigation, power, and flood studies increased the apparent power possibilities by more than 33⅓% in addition to other flood and navigation benefits.

Reporting on the Tennessee River above Chattanooga and its possibilities for run-of-river plants, Document No. 319 gives the following (in terms of installed capacity):

Dam No. 29	36 000 h.p.
Dam No. 30	58 000 "
Total	94 000 h.p.

In addition, it states, "in the entire stretch of the river between Knoxville and Chattanooga, 188 miles long, there appear to be no other favorable power sites."

Concerning the same section, Document No. 463 gives the following:

Coulter Shoals	75 000 h.p.
Marble Bluff	70 000 "
White Creek	150 000 "
Soddy	150 000 "
Sherman	45 000 "
Total	490 000 h.p.

* 67th Cong., 2d Session, pub. in May, 1922.
† 69th Cong., 1st Session, pub. in June, 1926.

From this it will be noted that on this section of the Tennessee combined navigation, power, and flood studies increased the apparent power possibilities by more than 400%, in addition to the other important flood and navigation benefits. It developed, for power, the entire head between Knoxville, Tenn., and Chattanooga.

Prior to the survey, power companies were interested only in Dams No. 3 and No. 21 (Document No. 319 list) on the Clinch River, and this interest was largely academic. When Document No. 463 was published, power companies had already filed with the Federal Power Commission applications for preliminary permits for the nine sites listed for both the Clinch and Tennessee Rivers, as well as for others.

In conclusion, it is submitted that the effect of combined power and flood use of storage should never be prejudged, but each case should be investigated carefully and judged strictly on its merits.

CHARLES F. BROOKS,* ESQ. (by letter).—Dr. Frankenfield presents the average rainfall by water-sheds in the Mississippi Basin and details the rainfalls by months prior to some of the great floods. Comparing 1927 with 1922, he indicates that the first four months of both years had about the same excess of rainfall, but that the greater flood in 1927 was due to the greater excess in April. Such being the case, it seemed worth while to study these April rains in relation to the weather map situations, both local and general. The results of a small investigation on these lines jointly by Nesbitt H. Bangs, Graduate Student, Clark University, and the writer are summarized herewith.

Although the Mississippi flood, like the New England flood (1927), was caused by the importation of vapor from tropical waters precipitated in excessive amounts over a region that is extended in area, it was not the result of a single importation. Rather was it brought about by a series of such importations extending back to the beginning of August, 1926, but culminating in the astonishing amounts of rain which fell in the Middle Mississippi Valley in April, 1927.

For a large area comprising Southern Illinois, Eastern Kansas, Eastern Oklahoma, Western Tennessee, Missouri, Arkansas, and Louisiana, the rainfall averaged more than twice the normal. The precipitation on parts of this region was far more than this, as, for example, Arkansas with a rainfall of 12.93 in. compared with a normal of 4.88 in., and Western Tennessee with a rainfall of 11.35 in. compared with a normal of 4.40 in. Curiously enough, the State of Mississippi as a whole experienced only the normal rainfall, but certain stations in the northwestern counties recorded rainfall far in excess of the average, notably Greenville, with a rainfall of 13.67 in., an excess of 9.06 in. above the normal. Table 37 shows the rainfall in the various States and the departure from normal.

The rainfall for the month was not only intense, but was also remarkably persistent. From April 1 to April 22, rain fell in considerable amounts every day in some part of the region. Only on April 2 and 3 did Arkansas,

* Prof. of Meteorology and Climatology, Clark Univ., Worcester, Mass.

for example, report no precipitation during this period, but on those days rain was falling in Illinois. Table 38 shows the periods when the rainfall was general and copious in the different sections of the region.

TABLE 37.—RAINFALL, IN INCHES, APRIL, 1927.*

	April, 1927.	Normal.	Departure.
Arkansas	12.93	4.88	+8.05
Western Tennessee	11.36	4.40	+6.96
Eastern Oklahoma	9.48	5.47	+4.01
Missouri	8.46	3.73	+4.73
Southern Illinois	8.43	3.92	+4.51
Eastern Kansas	7.40	3.05	+4.35
Louisiana	6.85	4.70	+2.15
Mississippi	5.12	5.19	+0.07

* From *Climatological Data*, April, 1927.

Scrutiny of this table shows five well-defined periods of rainfall: the 1st, 5th to 9th, 7th, 11th to 16th, 18th to 21st, and 29th. A study of the weather maps for these periods indicates a close analogy between the conditions causing the rain on the 1st and 29th, the rainfall on the 5th and 18th to 21st, and indicates that the rainfall during the long period from the 7th to 16th was caused by a pressure distribution that remained persistently the same throughout the period.

TABLE 38.—RAINFALL PERIODS.

Days of month.	APRIL.																												
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30
Arkansas	*	.	.	.	*	*	*	*	*	.	*	*	*	*	.	.	.	*	*	*	*	.	.	.	.	.	.	.	.	.
Western Tennessee	*	.	.	.	*	.	.	*	*	.	*	*	*	*	*	*	.	.	.	*	*	*	.	.	.	.	.	.	.	*
Eastern Oklahoma	.	.	.	.	.	.	*	*	.	.	*	*	*	*	*	*	.	*	*	*	*	.	.	.	.	.	.	.	.	.
Missouri	*	.	.	.	.	.	*	*	*	.	*	*	*	*	*	*	.	*	*	*	*	.	.	.	.	.	.	.	*	.
Southern Illinois	*	.	.	.	*	.	*	*	*	.	*	*	*	*	*	*	.	*	*	*	*	.	.	.	.	.	.	.	*	*
Eastern Kansas	*	.	.	.	.	.	*	*	*	.	.	*	*	*	*	*	.	*	*	*	.	.	.	.	.	.	.	.	.	.
Louisiana	.	.	.	.	*	*	(north)							*	*	*	.				*	.	.	.	.	.	.	.	.	.

Any conditions that will bring about the rapid ascent and consequent rapid cooling of warm moist air will cause heavy rain, and these conditions were particularly suitable during most of April. Pressure conditions over the United States and the Atlantic Ocean, as shown by maps,* were such that the southerly and southeasterly winds blowing up the Mississippi Valley originated, not merely over the Gulf of Mexico, but far to the southeastward over the Caribbean; hence they were warmer at the start, and passing as they did over such a great expanse of the warm Equatorial Current were given an amount of water vapor that was greater than normal through a considerable depth of winds. Arriving in the Mississippi Valley, these moisture-laden

* In *Monthly Weather Review*, April, 1927.

winds were forced to rise first over the mountains of Arkansas, and, more important, over great wedges of cold dense air pressing in from the northeast, north, and the northwest. No wonder the rainfall was heavy.

Given then this general picture of a vast flow of air from tropical waters meeting cold dense air from the higher latitudes, it is interesting to note under what local conditions heavy rainfall ensued. In general, the rainfall was of three types: (*a*) That which occurred in warm, southerly surface winds on the southeastern quadrant of a low-pressure area; (*b*) the warm-front type; and (*c*) the cold-front type.

(*a*).—In this case the lower wind coming directly from the tropics becomes warmer much more rapidly than the upper wind, which is usually from some westerly direction. Instability and consequent violent overturning of the surface air results, forming thunderstorms. At times, when the wind aloft becomes much colder from its source in a cold southwestern quadrant of the low-pressure area, the unstable overturning of the warm southerly wind below may reach cloudburst and tornadic violence.* The great number of heavy thunderstorms and tornadoes over the region bears witness to the frequency of this occurrence. Of forty tornadoes reported during the month nearly all occurred over the region discussed.

(*b*).—The second type of rainfall which fell when the warm southerly wind was forced to rise over a cold northeasterly surface wind flowing out from an area of high pressure to east and northeast, the so-called "warm-front" type, is usually a combination of general ascent and instability rainfall.

(*c*).—The third type occurred when the front of an energetic high pressure area pushed southeastward with cold northwest winds under-cutting or over-cutting it and forcing the warm, moist surface air upward. This is called the "cold-front" type and the rain falls with northwesterly winds at the surface and with rapidly falling temperatures. Where the over-cutting runs 50 or more miles in advance of the cold front at the surface, this type merges with the violent aspects of the southerly type.

What were the weather map situations of April that brought about the rainfall periods previously noted? On the morning of the 1st a low-pressure area of considerable intensity was centered over Missouri and Illinois and heavy rain fell with southerly winds over Missouri, Illinois, and Western Tennessee, averaging 1 to 2 in. in Southern Illinois. Here was an example of the first type; that is, heavy showers in southerly winds caused by overturning of the warm moisture-laden air.

On the 5th, an area of low pressure that had been centered over Oklahoma the preceding day had moved northeasterly to Lake Superior, and a wedge of cold air with moderately high pressure had cut in to the southwest of the low-pressure area. Heavy rain fell with northwest winds in Mississippi, Missouri, Arkansas, and Louisiana (Shreveport, 2.82 in.). This was an example of the cold-front type caused by the crowding in of cold air to displace the warm moist air of the preceding southerly winds.

* "Elementary Meteorology," by W. M. Davis, Bost., 1894, p. 258.

On the 7th an area of high pressure of great magnitude appeared over the Province of Ontario, Canada, and the Lake region. An ill-defined area of low pressure was centered over Texas, and this type of map persisted until the 16th. A great mass of cold air, constantly reinforced, was moving southeastwardly from Manitoba, Western Ontario, and the Lake region. Throughout this period there was a constant conflict between the cold surface northeast winds flowing from the high-pressure area and the warm southerly winds flowing northward in the Mississippi Valley on the easterly side of the low-pressure area in Oklahoma. It is difficult to indicate the exact point at which the northeasterly winds ceased and the southerly winds commenced, but at St. Louis, Mo., and Cairo, Ill., there were apparently seven days of rain with northeasterly winds; at Memphis, Tenn., four; and at Vicksburg, Miss., one, showing that the line of maximum rainfall with northeasterly winds was somewhere between Cairo and Memphis.

North of this line the surface wind was northeast and the rainfall was of the warm-front type, caused by the warm wind ascending above the cold wedge flowing out from the northeast high-pressure area; but south of the line, the rain that fell was of the instability type, (*b*).

An interesting variation of these types occurred during this period, especially on April 13-14, when the Oklahoma low-pressure area shifted slightly to the northward, as a tongue of cold air swept over Texas. Heavy rains of the cold-front type fell with northwest winds over Texas on these days (Dallas, 2.06 in. on the 13th). In the period, April 10 to 14, twenty tornadoes occurred, most of them in Texas. When the cold front, now in the free air, reached New Orleans, La., on the 15th, 14.00 in. of rain was precipitated.

On the 19th an energetic area of high pressure commenced to speed southeastward from the Northern Rocky Mountain region toward the Gulf of Mexico, and for the next three days (19th, 20th, and 21st) during its passage, extremely heavy cold-front rains (5 to 12 in.) fell in Arkansas.

The final period of heavy rain came on the 29th when an area of low pressure very similar to that of the 1st passed eastward over Missouri. The rainfall was particularly heavy in Illinois, Missouri, and Tennessee (on the 30th).

Thus, from the general description just presented, it can be seen that the rainfall periods bore certain resemblances to one another. For instance, those of the 1st and 29th occurred with southerly winds; those of the 5th and 18th to 21st were of the cold-front type, falling with northwesterly winds; and those of the 7th to the 16th were generally of the warm-front type, falling along the line where the warm winds on the eastern side of the low-pressure area began to override the cold air from the high-pressure area to the eastward. An attempt has been made to separate the total precipitation at a group of selected stations into the amounts that fell with northeasterly, southerly, or northwesterly winds.*

* Acknowledgment is made to Mr. A. J. Henry, of the U. S. Weather Bureau, for the manuscript 12-hour precipitation charts, and to Mr. Thomas R. Brooks, of the Weather Bureau, for kindly copying a number of the 8.00 P. M. weather maps for April, 1927.

To assume that all the rain which fell during the previous 24-hour or 12-hour period fell with the wind from the direction indicated at the end of the period is possibly to stray appreciably from the actual facts, although the persisting character of the weather map situations from day to day in these rainy periods make the results seem probably significant.

Over the flooding rain region as a whole, as represented by nine stations, 43% of the 12-hour rainfall (8:00 P. M. to 8:00 A. M.) came in the region of southerly surface winds, 35% in the northeasterly sector, and 22% in the region of northwesterly winds. The corresponding 24-hour values are almost exactly the same, 42, 34, and 23 per cent.

It is interesting to note that the more northerly stations had the greater amount of rains of the warm-front type. Thus, St. Louis and Cairo had 48% and 41%, respectively, of this type, whereas Shreveport and Vicksburg had only 2 and 7 per cent. This was to be expected, considering the pressure distribution already noted; but whatever the type, it is obvious that the excessive rains were due to the large importation of vapor from tropical waters.

Conclusion.—The April contribution to the Mississippi flood was not a single paroxysmal rainstorm, such as produced the New England flood in November, 1927, but a number of heavy rains at frequent intervals over substantially the same region. The vapor for these rains came day after day in a great flow of air from Southern waters. Suffering little depletion on the coast, this air converged in the central portion of the Lower Mississippi Basin and was there also forced to rise not only over the mountains of Arkansas, but also over great barriers of denser air pouring out of the northeast, north, and northwest. Furthermore, as the frequent thunderstorms bear witness, so great were the temperature contrasts in short distances, both vertically and horizontally, that the warm humid air was forced violently to great heights. These several causes of ascent and, therefore, cooling and drying of enormous masses of humid air acted over practically the same regions for days at a time, and, in consequence, produced the tremendous rainfalls that were chiefly responsible for the great flood.*

William Gerig,† M. Am. Soc. C. E. (by letter).—Major Godfrey has presented an excellent paper on the subject of the improvement of navigation and flood control, and has correctly described the characteristics of the Lower Mississippi River, particularly during the low-water stages when navigation improvements are necessary.

A channel 9 ft. deep has been maintained during the past thirty years (since about 1890) by dredging between Cairo, Ill., and the mouth of the Red River. This channel is temporary and is dredged, on some of the bars or crossings, during each low-water period. The depreciation of the channel is caused primarily by material deposited on the bars from caving banks immediately above the troublesome crossing. If, therefore, the banks of the river are stabil-

* For a full discussion of the Mississippi flood see *Monthly Weather Review*, Supplement 29, by H. C. Frankenfield and others, summarized in *Monthly Weather Review* for October, 1927, pp. 437–452, 10 diagrams and photographs, and very briefly in *Science*, January 5, 1928, pp. 15–16.

† Senior Engr., Office of Chf. of Engrs., U. S. Army, Washington, D. C.

ized, the channels in the crossings will unquestionably become more fixed as the quantity of material carried by the river is considerably reduced. The river will then cease, to some extent at least, its "restlessness".

The low-water channel in certain localities, even after the banks are stabilized, will require some dredging or regulation in order to hold it in a definite location. The regulation can probably be done, to some extent, by contraction works of proper design. The writer believes that considerable experimental work should be done in order to develop, at a reasonable cost, an effective type of contraction works for the lower river.

Since 1883 many types of contraction works have been tried on the Mississippi River below Cairo. They consisted of permeable pile and brush dikes, abattis dikes, massive rock-filled brush dikes, sand dams, etc., all of which were either total failures or have proved to be only temporary. These types were successful on the Upper Mississippi and other rivers. However, if the banks are stabilized well in advance of the construction of contraction works, a proper low-water type will no doubt be developed.

It has been computed that the Mississippi River expends at the rate of 60 000 000 h. p. of energy or heat during great floods in its course from Cairo, to the Gulf of Mexico. Some of this energy is, of course, expended in the erosion of the river banks and in the transportation of material in suspension. If bank erosion is curtailed, the energy saved will possibly be utilized to deepen the channel. The writer, however, does not believe that such an improved low-water channel will have any marked effect on flood heights, except that they may be slightly reduced in floods where the conditions are similar to those of 1927. During that year the Mississippi River was at or above flood stage for many months prior to the great rainfall in the lower valley during March and April, and at no time did the river fall sufficiently to permit erosion of the bar channels that had been filled. Erosion of sand-bar obstructions in the channel begins soon after the stream has precipitated some of the material in suspension. Precipitation starts when the peak of the flood is reached and continues as the river falls, and as the river is relieved of the work of transportation, it applies this energy to providing a channel across the bar obstructions between the pools. The Mississippi River has one characteristic that is common to almost all streams whether large or small, that is, it is composed of a series of pools separated by shoals and rapids. This condition (high bars across the channel) has decreased the slope of the river to some extent, and at Cairo, and other points, has caused somewhat higher stages for a given discharge, than would have occurred under normal conditions or with stabilized banks.

Again, when the banks are stabilized, the depreciation of the channel below a crevasse will be decreased, for the reason that the river is not transporting such large quantities of material in suspension, its source of supply has been considerably curtailed, and, hence, there will be little difference between the effect on the low-water channel of a controlled spillway or weir and that of the uncontrolled spillway.

The writer agrees with Major Godfrey that some minor straightening of the low-water channel, as long as the general regimen of the river is not affected, may be advantageous in some localities. Careful study, however, should be given such proposals before they are undertaken, or otherwise the high-water slopes may be affected.

C. H. EIFFERT,* M. AM. SOC. C. E. (by letter).—Mr. McCarthy states the weight of dry forest litter at the Wooster, Ohio, Station to be from 1 827 to 17 545 lb. per acre and that it will absorb 1½ to 2½ times its dry weight in water. The weights were obtained in the fall before the leaves had fallen. The annual deposit of litter is estimated to be about 2 tons, dry weight. The average weight of dry litter per acre would be about 5 tons. Add to this the annual crop of dry leaves and there would be about 7 tons of litter per acre to absorb water during the following flood season. Assume the absorptive capacity of this litter to be two times its dry weight, or 14 tons per acre. This would amount to 450 cu. ft. of water, or 0.12 in. of rainfall. The Ohio Basin is estimated to be one-third forested. (The area that might be reforested, although it amounts to considerable in acres, will affect this proportion very little.) If 0.12 in. were absorbed by the forested area, it would amount to 0.04 in. over the entire Ohio Basin. The absorptive capacity of the litter when dry is assumed to be 14 tons per acre. It will never be dry during February, March, or April, when the great flood-producing rains occur. Assuming the quantities of litter stated to be the average, the net effect of the forest litter on run-off may be considerably less than 0.04 in. in the Ohio Basin in a great flood.

In the quotation from "The Relation of Forests and Water" by Zon, it is stated that forests retard the melting of snow in the mountains. Whether or not this retarding action helps to reduce floods is questionable. The forest in this case acts as an uncontrolled reservoir and may release its storage at the wrong moment.

It is very difficult to take the results of experiments such as those being made at Wagon Wheel Gap, Colo., in the White Mountains, or in Switzerland, and try to deduce therefrom possible results in the Mississippi drainage basin. These are small areas. Their geology, climatic conditions, altitudes, and geographical locations differ greatly from the parts of the Mississippi Valley contributing most of the flood run-off. The Swiss experiments were referred to in order to show the difference in run-off between forested and deforested areas during a violent summer storm. These experiments seem to be the most thorough and complete that have been made along this line, and the results for a period of thirteen years have been reported by Dr. A. Engler. Some of the conclusions reached are as follows:†

"The run-off due to heavy rainfalls of short duration as thunderstorms or cloudbursts is much less from the forested than from the deforested area, peak discharges from the forested area being from one-third to one-half of those from the deforested tract and the total run-off for this class of storms from the former is usually about one-half that from the latter.

* Chf. Engr., The Miami Conservancy Dist., Dayton, Ohio.

† The writer's translation.

"The run-off due to long continued rainfall is greatly affected by the water content of the ground prior to such rainfall. In case this water content is considerable the forest will have no effect, the run-off being the same from forested as from deforested areas.

"Over the entire period of 13 years the run-off factors for the two areas are practically identical, being 60.0% for the forested and 60.4% for the deforested area."

The writer wishes to emphasize the matter of the water content of the soil or its absorptive capacity by an illustration from the Miami Valley. This valley may be considered as deforested. During the storm of September 12 and 13, 1925, the average rainfall north of Dayton, Ohio, over an area of 2 525 sq. miles, was 3.20 in. The run-off was 0.04 in., or 1.25% of the rainfall. During the storm of March 19 to 21, 1927, the rainfall was 3.44 in. and the run-off resulting from it was 2.31 in., or 67% of the rainfall. The first-mentioned storm was of shorter duration and other factors being equal, would have given the greater run-off. The maximum stage reached at Dayton during the first-named storm was 2.9 ft. while during the latter it was 12.8 ft.

Unquestionably the difference in run-off between these storms was due to the difference in soil conditions. The storm of September, 1925, came at the end of the growing season and following a dry period during which the transpiration and evaporation had been abnormally great. At the beginning of this period there had been a deficiency in rainfall of about 5 in. since the first of the year. These factors combined to give the soil a tremendous storage capacity.

Preceding the storm of March, 1927, there had been little evaporation, and there was no growing vegetation to consume moisture. To November 1, 1926, there had been an excess of 5.79 in. in precipitation since the beginning of the year.

It seems to the writer that while the absorptive, retentive, and retarding effects of the forests have been greatly emphasized by those who advocate reforestation as a means of assisting in flood control, the same factors in regard to pastures, growing crops, or well cultivated fields, have been generally neglected or entirely disregarded. The comparisons made are too often between forests and denuded or barren hill and mountain sides. That part of the Mississippi Basin east of the 100th Meridian yields more than 95% of the Mississippi flood water. The part of this area which is not forested, consists principally, not of barren hillsides, but of well tilled farm lands which have a considerable capacity for retaining moisture.

While there may be a small surplus of farm land at the present time, no one would advocate returning good agricultural land to forest conditions. On the other hand, there can be no valid objections to reforesting the denuded hills, not for flood-control purposes, but for the products of the forest. Let forestry stand on its own merits. Even if it could be shown that forests might have an important influence on great Mississippi River floods, the areas which might be available for reforestation would not be great enough to alter materially present conditions.

The forest will undoubtedly prevent, to a large extent, the washing away of the soil from the hillsides, but this is not necessarily an argument in favor of reforestation for flood control. The Missouri experiments, mentioned by Mr. McCarthy, seem to show that it will require 437 years to erode the 7-in. surface layer from land planted with a rotation of corn, wheat, and clover, and 3 547 years to erode 7 in. from land in blue-grass sod. One may see gullies and other signs of erosion even in the forest. Will it take longer than 3 547 years to take 7 in. of soil from forested land?

Referring to Colonel Kelly's paper, the writer desires to call further attention to the possible effect of the Miami River flood-control works on floods in the Ohio River.

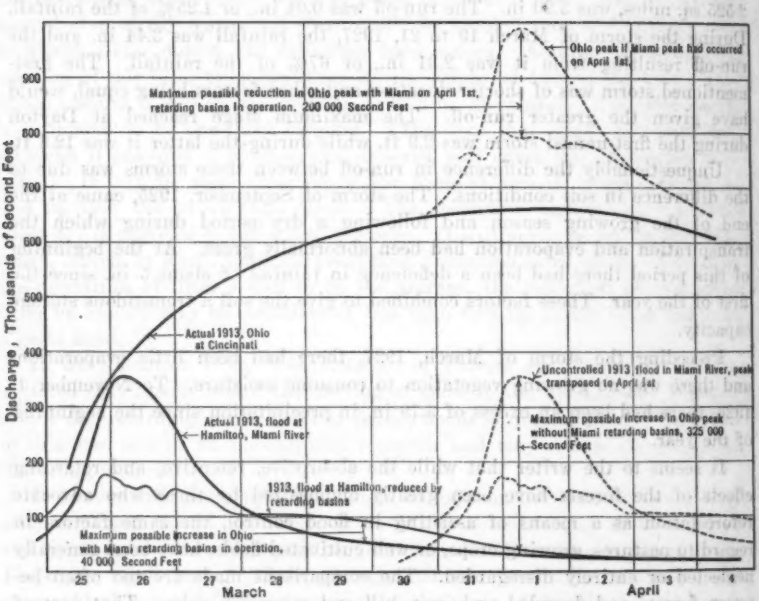

FIG. 46.—HYDROGRAPHS OF THE 1913 FLOOD.

In Fig. 46 are shown hydrographs of the actual 1913 flood in the Ohio, at Cincinnati, the 1913 flood in the Miami River, at Hamilton, a controlled 1913 flood, at Hamilton, and a 1913 flood in the Ohio with the peak in the Miami, occurring on April 1 instead of March 26. The actual figures at the mouth of the Miami River would exceed those shown herein. Information for completing hydrographs for both streams at the mouth of the Miami was not available; however, the curves shown answer the purpose of illustrating the point it is desired to emphasize. Comparing the curves for the actual and the controlled 1913 flood, at Hamilton, it is seen that the maximum excess of the controlled flood over the actual is 40 000 sec-ft. and occurs on March 29. The maximum possible increase in the Ohio, therefore, would be 40 000 sec-ft.

for such a flood and would occur if this stage of the Miami flood occurred simultaneously with the maximum stage in the Ohio. Under any other conditions the increase would be less than 40 000 sec-ft. Suppose, however, that the peak in the Miami, instead of coming on March 26, should synchronize with the Ohio peak occurring on April 1. The uncontrolled Miami River flood would then increase the Ohio peak by 325 000 sec-ft., but controlled as at present this increased peak would be reduced 200 000 sec-ft. In other words, in a flood like that of 1913 controlled by the retarding basins, the maximum possible increase in the Ohio River would be 40 000 sec-ft., while the maximum possible reduction might be as much as 200 000 sec-ft.

It cannot be safely argued that because there is no record of this combination, it cannot occur. There are no definite previous records of floods as great as that of 1913 on the Miami, or like that of 1927 on the Mississippi. Before they occurred, they were thought by many to be impossible. Yet certain combinations made them possible.

The writer does not wish this to be understood as an argument in favor of reservoirs on the head-waters of Mississippi tributaries. He simply desires to state a possibility which has not heretofore been emphasized.

SHERMAN M. WOODWARD,* M. AM. SOC. C. E. (by letter).—The paper by Colonel Kelly serves as an admirable illustration of the urgent necessity for the creation of a fact-finding commission, authorized to use ample time and money, to whatever extent may be necessary, in order to investigate and determine the best solution of the Mississippi River flood problem.

If engineers had accurate maps of actual important reservoir sites with data showing storage capacity at various depths; if they had detailed estimates of quantities and costs covering the dam, spillway, control gates, and flowage rights, all accompanied by complete flood hydrographs of the stream bearing the reservoir site and other information needed to show the time relation of the storage to the flood flow in other tributaries, and to the regions to be protected; if they had such actual information, it would be possible to discuss, much more satisfactorily, the methods used for determining the cost of reservoir control, the comparative value and merit of such control, and the reliability of the conclusions reached.

As it is, it may be profitable to consider briefly some methods used in such studies. The approximate studies already made seem to indicate that complete protection might be secured by levees alone at a cost of approximately $1 000 000 000, and that an equal degree of protection could be secured by reservoirs at a cost somewhat greater. The wisest and most economical protection would then undoubtedly be some combined plan using the most advantageous features of both methods. To work out the best combined plan requires careful detailed consideration of each feature and each reservoir separately. In such a study, to lump the effect of a number of reservoirs or to average them, would vitiate the results. The last or top foot of such a system of levees might cost $100 000 000. If a group of reservoirs were found, the construction of which would contribute to the flood protection at a more

* Prof. of Mechanics and Hydraulics, State Univ. of Iowa, Iowa City, Iowa.

advantageous or cheaper rate, this group should be included in the combined plan, and the expenditure on levees should be reduced accordingly. Even if only a single reservoir were considered advantageous, its effect being considered equivalent to 0.1 ft. on the height of the levees, the levee should be lowered that much, thereby reducing the expenditure for levees by perhaps $10 000 000 and using the needful amount for the cost of the reservoir. This should be the method of balancing to be used, notwithstanding the fact that perhaps the ultimate fixing of the levee grade remains finally a somewhat arbitrary matter, being placed empirically at some foot or half-foot mark. The point is that the comparative advantages and disadvantages of different features can be and should be determined with more precision than may be claimed for the finally adopted dimensions.

The estimates so far published (1928) for the construction of floodways are vague and uncertain. When these estimates are satisfactorily settled, the same plan of comparison may be equally well applied to them.

It would be highly desirable to build at least one large reservoir or one group of reservoirs conveniently situated, for flood protection on the Lower Mississippi, if merely as a demonstration or experiment and for the purpose of testing the operation of the reservoir during flood times. This is a field in which there is very little experience as a guide.

What Colonel Kelly calls dependable operation of flood-control reservoirs, is storage at a uniform rate for a period of 90 days, entirely irrespective of flood conditions on other tributaries, or any other existing condition. This is perhaps the least efficient system of operation that can be imagined and still be called a system of rational control. It seems probable that reservoirs close to the region to be protected can be operated three times as efficiently. Colonel Kelly bases most of his estimates of comparative value on averages of large groups of reservoirs. To determine the most economical combined system, the comparison should begin with the least expensive single reservoir measured in units of flood protection, and then should proceed regularly toward the more expensive. Taking all these items into consideration, it appears probable from Colonel Kelly's figures that a considerable number of reservoirs will be desirable and economical in a combined system for flood protection. It is impossible to determine this point satisfactorily or conclusively until more detailed and accurate data are available.

The other point to be considered is the question of the advantage of close proximity of the reservoir to the area to be protected. It is a great advantage to have some of the reservoir storage near the protected area, but it should be understood that it is not necessary or particularly beneficial to have all the reservoirs close together. This can most easily be made clear by a simple example. Assume two equal reservoirs on the same or different tributaries with adequate flow, Reservoir A being close to the protected area and Reservoir B, two weeks distant. It can be shown that, under rational control, they can in general be operated as efficiently as if both of them were as close to the protected area as Reservoir A. Assume that on a certain date conditions make it desirable for the flow below Reservoir A to be reduced by a constant amount, such that when stored in Reservoirs A and B it would just

fill both of them in the eight weeks. Let x = one-eighth of the capacity of Reservoir A. Then, to effect the desired protection below Reservoir A, $2x$ is the quantity of water per week by which the flow below it is to be reduced. The operation of the controlled reservoirs would be as follows.

On the given date, begin storing water in Reservoir A at the rate of $2x$ per week. This would reduce the flow below Reservoir A by the desired amount. On the same date begin storing water in Reservoir B at the rate of $\frac{4}{3}x$ per week. At the end of 2 weeks the reduced flow below Reservoir B would reach that of Reservoir A. At this time change the rate of storage in Reservoir A to $\frac{2}{3}x$ per week. This rate of storage combined with the reduced rate of flow from Reservoir B would maintain the desired reduced flow below Reservoir A. At the end of 6 weeks from the beginning, Reservoir B would be full, and the entire stream flow would be allowed to pass, but this flow would not reach Reservoir A until 2 weeks later, that is, at the final end of the 8-weeks period. At this final date Reservoir A would also be completely filled, having stored at the rate of $2x$ for the first 2 weeks and at the rate of $\frac{2}{3}x$ for the next 6 weeks, making a total final storage of $8x$ in each reservoir.

This simple example is only to illustrate the general method. Whatever the sizes and locations of the various reservoirs in a system, they can be similarly operated in co-ordination with each other with perfect elasticity to the capacity of the stream supplying each reservoir.

W. J. Schlick,* M. Am. Soc. C. E. (by letter).—The careful study made by Professors Woodward and Nagler, furnishes valuable evidence on the much debated question of the effect of drainage development on flood flows. Although the available data are not extensive, the definiteness of the indicated conclusions warrants their general acceptance. It is doubtful if the effect of drainage will be more pronounced on any of the streams in the Mississippi water-shed, so there seems to be no question of the accuracy of the conclusions in the paper as applied to floods of that great river, particularly since only a small portion of any major flood flow can come from an area which is drained, or in which any extensive drainage development is economically probable.

It seems that, if there are any inaccuracies in the conclusion, these are the results of insufficient data and apply to smaller water-sheds in which lateral under-drainage is, or may become, more nearly complete. The water-shed of the Des Moines River above Fort Dodge, Iowa, is of such character that a very large proportion of it probably will be underdrained eventually. The resulting effects of this drainage development on flood flows, and particularly on maximum flood flows, are problematical but are worthy of consideration.

In 1920 the Iowa Engineering Experiment Station instituted a study of the discharges from several of the larger drainage districts of the State. In two of these districts (with drainage areas of approximately 25 and 35 sq.

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miles, respectively), the outlet drains were just being completed. The fact that the rainfall during the spring and summer months has not been excessive, and that there has been an economic depression in the agricultural States, has so retarded the installation of farm under-drainage systems that the flow records obtained to date (1928) represent very largely the surface run-off admitted to the open ditches directly and through inlets in the district tile lines. The curves of "surface flow", Fig. 47, are for one of these districts and a lateral area of it.

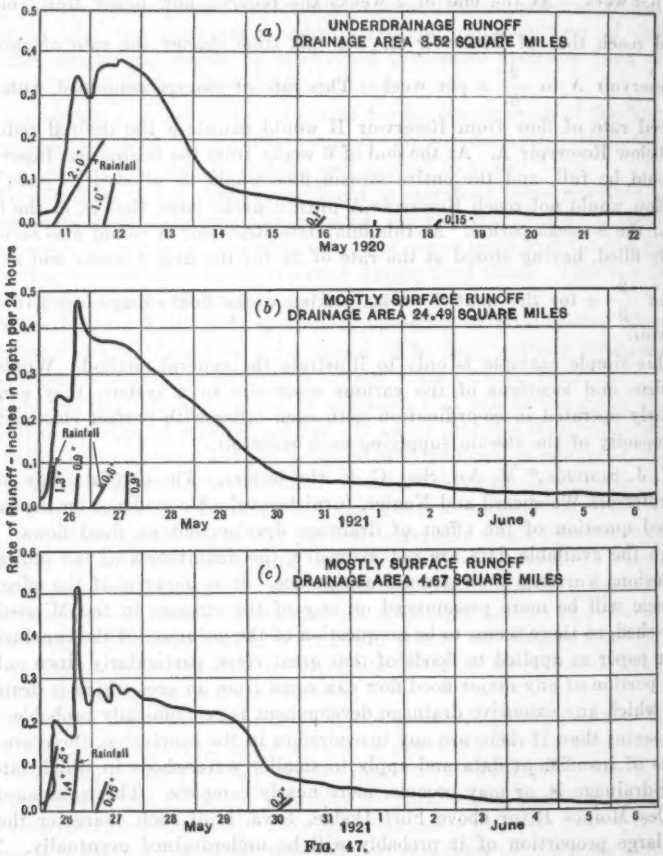

FIG. 47.

Another of the districts studied was rather completely underdrained, with no provisions for admitting surface run-off. The discharge from this system is entirely under-drainage flow, except for the surface run-off of a small area immediately adjacent to the outlet.

The shapes of the rate-of-run-off curves shown on Fig. 47 are illustrative of those obtained during this study. They present some interesting possibili-

ties in connection with the study of the effects of drainage on flood flows. The curves for the "under-drainage" area are always flatter and the peaks less sharp, than those for the "surface run-off" areas. Neither of the curves represents the run-off from the maximum storms that reasonably may be expected, although the maximum rates shown are as high as any during the period from 1920 to 1927, inclusive. Only one of these areas is in the water-shed of either of the streams studied by Professors Woodward and Nagler, but they are typical, in a general way, of the flatter areas of both those water-sheds.

The available data do not warrant any definite conclusion as to what effect the extensions of under-drainage may have on the correctness of the conclusions of Professors Woodward and Nagler. The percentage of the water-shed of the Des Moines River that probably will be underdrained is unknown, but it is doubtful if it will exceed 75% of the water-shed above Fort Dodge, 50% of that above Des Moines, and 25 to 33% of that above Keosauqua. Since the present drainage development has had no determinable effect on flood flows, it seems reasonable to expect that the effect of future under-drainage development on the flood flow at Des Moines and Keosauqua will not be material. However, the shapes of the curves do indicate the possibility of such an effect, especially on the medium high floods from the smaller water-sheds.

The present-day limits of the economical maximum intensity of under-drainage are such that the storms producing the maximum flood flows usually cause some surface run-off from underdrained areas. Also, the tile lines of nearly all drainage-district systems are now provided with inlets to hasten the removal of surface water. These two facts will tend to minimize any effect under-drainage may have on flood flows, particularly maximum flood flows.

S. L. Moyer,* Esq. (by letter).—The wide divergence in run-off behaviors under the current conceptions of the relation of run-off to rainfall, is most effectively disclosed by an examination of the diagrams presented in connection with the paper by Messrs. Woodward and Nagler. This fact seems to indicate a very real need for a rational evaluation of the accidental influences which give rise to such a haphazard showing. This apparent need affords a timely opportunity to present an entirely different conception of the nature of the problem in hand.

The method of attack herein outlined assumes that there is some general law governing the accidental occurrence of storms, and that the chance for such events arises out of basic causes in combination, after the fashion of the "hit-or-miss" fall of the dice or cards.

As distinguished from the current theory of probabilities, this development does not assume that rainfall magnitudes are the result solely of causes in summation; but, on the contrary, it is conceived that these events may also be influenced by factors (dice of destiny) combining by multiplication, rather than summation, of values.

In common with other probability methods, time and count are herein assumed to be one and the same thing; and time in its totality or eternity is taken as unity, or 100 per cent. The extent of time, or count, is measured

* Civ. Engr., Montevideo, Minn.

in relation to the opportunity offered for the occurrence of magnitudes, without regard to the sequence of events; and the procession of time is viewed as to its content in event magnitudes, instead of in the ordinary terms of the chronological order of occurrence.

The words, expectancy and probability, are perhaps unfortunate in that they seem to suggest an element of uncertainty; but an expectancy examination of data, in its recognition of the accidental elements and in its attempt to assay the chance, elects to face a situation which other methods ignore. Once the fate factors are given proper weight, a surprising uniformity of results may be achieved.

An expectancy analysis was made, based on the data given for monthly rainfalls and run-offs for April, May, and June, during the two periods before and after drainage, covering the basins of the Des Moines River above Keosauqua, and of the Iowa River above Iowa City, Iowa. This single showing cannot be said to prove conclusively the validity of the approach to the problem in question. However, the apparent harmony of the results obtained from this analysis does seem to tend to prove that, on the expectancy average: (1) run-off is a proportion of rainfall after deducting a constant loss factor; (2) the constant loss factor is little, if at all, affected by agricultural drainage; and (3) as applying to the area actually subjected to drainage improvements, agricultural drains of the Iowa type have apparently effected a reduction in the run-off proportion to the extent that the value after drainage is about 40 to 45% of the original value before drainage.

An ordinary rain-gauge has a catchment area of (roughly) one hundred-millionth part of a square mile. In the case of the Des Moines and Iowa River basins, on which Weather Stations are fairly numerous, there seems to be about one significant rainfall record to a section of 300 sq. miles, or the gauge sample of the rainfall is about one thirty-billionth part of the section it presumes to represent. The gross inadequacy of such representation as evidence of what happens in a given storm on the 300-sq. mile section, may perhaps best be seen by comparison with a car of wheat.

An ordinary freight car transports about 1 500 bushels, or 90 000 lb. of wheat (1 440 000 oz.). An ounce of wheat contains somewhat less than 2 000 kernels; so a carload of wheat is made up of a scant 3 000 000 000 kernels. Assuming a sample of the wheat in proportion to the rainfalls in question, 1 kernel must be taken as a fair sample of 10 or 12 carloads, if the rainfall samples are fair, and the absurdity of the representation becomes self-evident.

On the other hand, it is a matter of common observation that different storms travel in different ways, so that during the course of a few years, the rainfall record at the gauge will include practically all the variations in intensity that fall to the lot of the whole section which the gauge represents. Therefore, the gauge record does fairly represent the trend of magnitudes for the whole section, and since expectancy has to do with the trend of magnitudes, the sample is fair for expectancy purposes.

It is obvious that run-off records fairly represent the whole of the events, either for a given storm or for the trend of storm magnitudes. The limita-

tions of mathematics demand that the association of values in any given operation be confined to subjects and predicates displaying uniformity in logical kind or type. This restriction, which is the result of extensive mathematical experience, therefore, suggests that the only fair basis for the relation of run-off to rainfall, is found in that aspect under which rainfall samples appear to be fair, or in the expectancy viewpoint.

In its essence, expectancy or probability is an assay of time as to its content in magnitudes. The writer conceives time to be the opportunity for the occurrence of events. An event may be said to be a manifestation of activity, a behavior in the material, or a phase of transition, which occurs, happens, befalls, takes place, or comes to pass in a series, a parade, or procession of incidents.

The monthly rainfalls and run-offs for the Des Moines and Iowa River basins above Keosauqua and Iowa City, given in Table 31, under the expectancy conception, may be regarded as events of the same kind or type with respect to the time lapse in which the magnitudes have accumulated. Some of the phenomena, however, are subject to widely different types of seasonal influence, and, therefore, may be said to be unfair in association. For this reason, all the magnitudes in Table 31 have been discarded, except those covering the months of April, May, and June in each of the years listed.

While these three months are not exactly of uniform seasonal kind or type, any difference is progressive and may be fairly regarded as reducible to expectancy. The inclusion of more than one month for each year of the record is not valid as to expectancy over a period of years, but this sort of representation is believed to result in assemblies of rainfall and run-off data which are logically comparable.

Even by this means of expanding the expectancy series by inclusion of several months in a given year, the remainder of the data given by the authors is entirely too scant for analysis by this method, so it must also be disregarded. Maximum 24-hour magnitudes for each of these months might be compared, but the writer prefers to have these data compiled by one who is a stranger to this system of analysis, in order to avoid any suspicion of "colored" statistics.

With fair samples on hand, observations to be used for expectancy purposes may next be graded, as sizes are screened and sorted in sand and gravel analyses. The accidental occurrence of the sizes in a gravel deposit, as to arrangement in place, is important in so far as it affects the picking of a fair sample, but after such sample has been secured, the original order of deposit has no particular significance. Similarly, the sequence of magnitudes in the time parade is of no consequence, after the expectancy samples have been selected, and the grading of the samples is effected simply by arrangement of the observations in the order of their sizes.

For the expectancy assay of events, no division into arbitrary classes, as in gravel analyses, is necessary because the analysis of the magnitude content of time is accomplished through the ratio of the count of observations,

found above and below any given stage, or by the odds of the number of items of data exceeding and short of the position in question.

Every line of attack on the probability problem, including even the classic Gaussian law of error, attempts to evaluate the content of time in its eternity; and as this can never be known exactly, each such effort to solve the mystery must of necessity be merely an empirical approximation of magnitude trends. It is the writer's belief that the method herein presented, represents a definitely practical step in advance, affording a broader insight into the various aspects of the subject than any hitherto proposed. The broad field to which the idea may be applied, if it proves sound, warrants searching and critical study.

As the mathematical mechanism is an empirical development, the practical operation will be shown before attempting to state the logical reasoning that may be said to support the writer's analytical expectancy charts (Figs. 48 to 52).

Notation.—

Let F = the frequency, or the average interval in the count of events between magnitudes exceeding any given stage.

P = the probability, or proportion of the count of events exceeding any given stage of magnitude.

N = the numerical designation of an observation when the expectancy series is arranged and observations are numbered in the order of magnitude, beginning with the least.

T = the total number of observations in the expectancy series.

d = any position on an arithmetical base scale covering a range from zero to unity, or covering time in its totality.

v and z = two arbitrary variables influencing the form and position of the scale of F (Fig. 48) in reference to the basic and fixed scale of d, and by means of which data trends may be reduced to an approximate straight line for one-half or more of the range of the trend in a given approximation.

e = the tangent of slope for any straight line on a chart (Fig. 48) having the d scale as a horizontal base and magnitudes as vertical arithmetical ordinates; or the difference in magnitude on the straight line between the two limits of the d scale.

c = the ratio of the magnitude on the straight line at the upper limit to e; or that position on the d scale where the straight line intersects zero magnitude or would intersect zero magnitude if the d scale were extended.

Q = the magnitude for any point on the straight line.

A = the proportion of catchment area unaffected by drainage improvements.

B = the run-off proportion for the area, A, after deducting constant loss factor.

C = the proportion of catchment area affected by drainage.

D = the run-off proportion for the area, C, after constant loss deduction.

E = the run-off proportion for entire area after drainage, the constant loss factor being first deducted.

For the purpose of assigning logical frequency values to the various observations in an expectancy series of data, as a guide to the plotting of the magnitudes on the writer's analytical expectancy chart, it can be demon-

strated that Equation (1) produces values that are consistent, on the average, for both long and short series:

$$F = \frac{T + 2\left(\frac{T-1}{T+5}\right) - 1}{T + \left(\frac{T-1}{T+5}\right) - N} \quad \ldots\ldots(1)$$

Expressing the numerator of Equation (1) by a, and assuming,

$$b = T + \frac{T-1}{T+5} \quad \ldots\ldots(2)$$

Equation (1) may then be written,

$$F = \frac{a}{b - N} \quad \ldots\ldots(3)$$

or, since P is the inverse of F,

$$P = \frac{b - N}{a} \quad \ldots\ldots(4)$$

Equation (4) is not essential to the distribution of data in this analysis, but it is introduced in order that a clear conception may be had of the relation to the ordinary, current conceptions of probabilities.

As a general equation by means of which the law of chance curvature for occurrences of a given kind or type may be asserted, the following empirical expression is offered:

$$d = \frac{v}{F} + \frac{(1 - v)\ (1 + z)}{F + z} \quad \ldots\ldots(5)$$

The form of Equation (5) is such that when $F = 1$, $d = 1$, and when $F = \infty$, $d = 0$. The limits of the d scale, therefore, always include infinite time or eternity. Writing Equation (5) in a form expressing the trend of rainfall and its consequences as to magnitudes expected,

$$d = \frac{3}{10\,F} + \frac{35}{F + 49} \quad \ldots\ldots(6)$$

Equation (6) also approximates the curve form for four chance elements falling in accidental combination and influencing event magnitudes by multiplication of cause values.

The magnitude for any point on the straight line shown in Fig. 48 is determined by the formula,

$$Q = (c - d)\ e \quad \ldots\ldots(7)$$

The writer's expectancy charts for rainfall and run-off (Figs. 48 to 52) are designed under and portray the law of chance expressed by Equation (6). By means of Equation (3), a series of observations of phenomena which are truly representative of the chance, may be assigned a logical frequency on the diagram against the F scale and plotted in proper frequency position. The graphic representation of the magnitudes may then be approximated by a straight line, and an equation of the expectancy representing the straight-line reconcilement of the data, may be derived under the principles of Equation (7).

In Table 39, the rainfall and run-off magnitudes listed in Table 31 and covering the Des Moines River above Keosauqua and the Iowa River above Iowa City during the months of April, May, and June, for which both rainfall and run-off magnitudes are listed, are arranged in the order of their respective sizes. These magnitudes have then been numbered in the same order and the frequency, F, has been computed as shown by the F values in Table 39, utilizing the principles of Equation (3).

TABLE 39.—FREQUENCY AND DATA.

Des Moines River, Keosauqua, Iowa.						Iowa River, Iowa City, Iowa.					
Before Drainage.											
N.	*F.*	Month.	Rainfall, in inches.	Month.	Run-off, in inches.	*N.*	*F.*	Month.	Rainfall, in inches.	Month.	Run-off, in inches.
1	1.07	April, 1906	2.40	May, 1906	0.52	1	1.07	April, 1906	2.29	May, 1904	0.58
2	1.235	May, 1906	3.18	May, 1904	0.61	2	1.235	June, 1903	2.45	May, 1906	0.64
3	1.458	May, 1904	3.22	April, 1905	0.66	3	1.458	April, 1905	2.91	April, 1905	0.70
4	1.778	April, 1905	3.42	June, 1905	1.00	4	1.778	May, 1906	3.03	April, 1904	0.82
5	2.28	June, 1903	3.63	May, 1905	1.04	5	2.28	April, 1904	3.54	June, 1905	1.03
6	3.18	April, 1904	4.37	April, 1906	1.14	6	3.18	May, 1904	3.69	April, 1906	1.69
7	5.25	June, 1905	5.66	April, 1904	1.25	7	5.25	June, 1905	4.59	May, 1905	1.73
8	15.	May, 1905	5.76	June, 1903	2.69	8	15.	May, 1905	7.12	June, 1903	2.59
After Drainage.											
(Area, 32.7% Drained).						(Area, 46.5% Drained).					
N.	*F.*	Month.	Rainfall, in inches.	Month.	Run-off, in inches.	*N.*	*F.*	Month.	Rainfall, in inches.	Month.	Run-off, in inches.
1	1.05	April, 1918	1.76	April, 1918	0.10	1	1.05	April, 1923	1.60	April, 1918	0.23
2	1.133	April, 1923	1.88	May, 1918	0.31	2	1.133	May, 1923	2.48	May, 1922	0.28
3	1.23	May, 1919	2.62	May, 1923	0.31	3	1.23	April, 1918	2.70	May, 1922	0.58
4	1.345	May, 1920	2.67	April, 1923	0.48	4	1.345	May, 1919	3.07	April, 1921	0.63
5	1.484	April, 1921	2.84	April, 1921	0.57	5	1.484	April, 1922	3.09	May, 1921	0.73
6	1.655	May, 1923	3.01	May, 1922	0.57	6	1.655	May, 1920	3.65	April, 1922	1.20
7	1.868	April, 1922	3.20	May, 1921	0.61	7	1.868	April, 1921	4.11	April, 1923	1.26
8	2.149	May, 1922	3.57	April, 1922	1.00	8	2.149	May, 1922	4.11	May, 1918	1.30
9	2.526	April, 1920	4.31	April, 1920	1.24	9	2.526	May, 1921	4.31	April, 1920	1.50
10	3.065	May, 1921	4.89	May, 1920	1.27	10	3.065	April, 1920	4.31	May, 1920	1.54
11	3.90	June, 1918	5.08	June, 1918	1.35	11	3.90	April, 1919	5.14	April, 1919	1.70
12	5.35	April, 1919	5.14	May, 1919	1.35	12	5.35	June, 1919	6.43	June, 1919	1.74
13	8.53	May, 1918	6.41	April, 1919	1.45	13	8.53	June, 1918	7.53	May, 1919	1.83
14	21.	June, 1919	7.05	June, 1919	1.67	14	21.	May, 1918	8.32	June, 1918	3.90

Based on the magnitudes and frequency values of Table 39, graphic distribution and representation of the data have been accomplished as shown in Figs. 48 to 51, inclusive.

Referring to Fig. 48, after plotting the data in accordance with the computations of frequency, these magnitudes have been approximated by a straight line for both the rainfall and run-off data. From the scales of this diagram,

values of the factors in Equation (7) have been derived as shown in Column (2) of Table 40. This process has been repeated for Figs. 49, 50, and 51, and the values have been derived as shown in the other columns of Table 40.

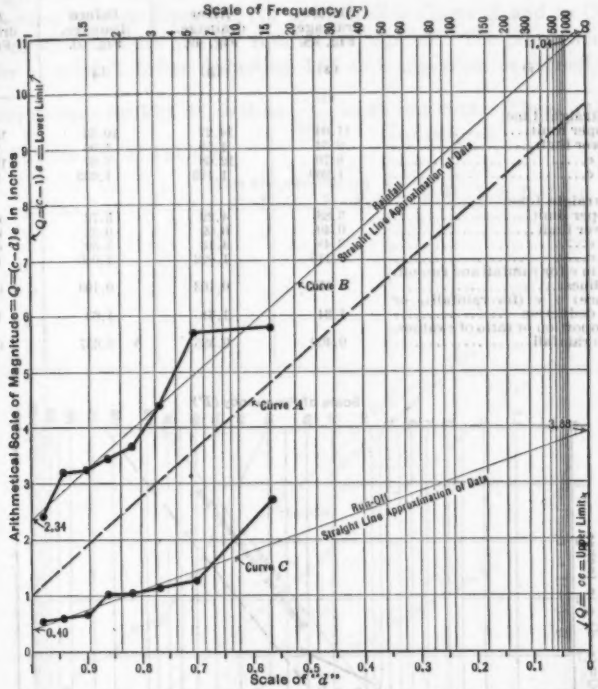

FIG. 48.—DES MOINES RIVER, KEOSAUQUA, IOWA, BEFORE DRAINAGE.

The approximation of the observed magnitude by a straight line in each of these cases is somewhat a matter of judgment, but from the very nature of the data arrangement, erratic magnitudes must tend to fall in the main in the upper or lower portions of the series. The magnitudes in the middle section are thus seen to be entitled to the greatest weight in such graphic reconcilement. In all eight of the series thus reconciled, no adjustments whatever have been made subsequent to the final computation of results.

In Table 40 (Column (2)), beside the item "Constant Deduction", appears the value, 1.34. In Fig. 48, a broken straight line (Curve *A*) is drawn parallel to and at a magnitude of 1.34 below the straight line for rainfall (Curve *B*). The value 1.34 is such that Curve *A* will intersect the straight line for run-off (Curve *C*), when both are extended, at the zero position on the scale of magnitudes. From this, it will be seen that after the deduction of 1.34 in. from rainfall, on the expectancy average, the remainder is related to run-off by proportion.

TABLE 40.

(1)	Des Moines River. Before drainage, Fig. 48. (2)	Des Moines River. After drainage, Fig. 49. (3)	Iowa River. Before drainage, Fig. 50. (4)	Iowa River. After drainage, Fig. 51. (5)
Rainfall, Straight Line:				
Q for upper limit	11.04	14.27	10.33	15.74
Q for lower limit	2.34	1.59	1.96	1.94
Value of e	8.70	12.68	8.37	13.80
Value of c	1.269	1.125	1.233	1.141
Run-off, Straight Line:				
Q for upper limit	3.88	4.22	5.72	6.71
Q for lower limit	0.40	0.09	0.22	0.12
Value of e	3.48	4.13	5.50	6.59
Value of c	1.115	1.022	1.040	1.019
Difference in c for rainfall and run-off, straight lines	0.154	0.103	0.193	0.122
c (difference) $\times$ e (for rainfall), or constant deduction	1.34	1.31	1.62	1.68
Run-off proportion or ratio of e values, run-off to rainfall	0.400	0.326	0.657	0.477

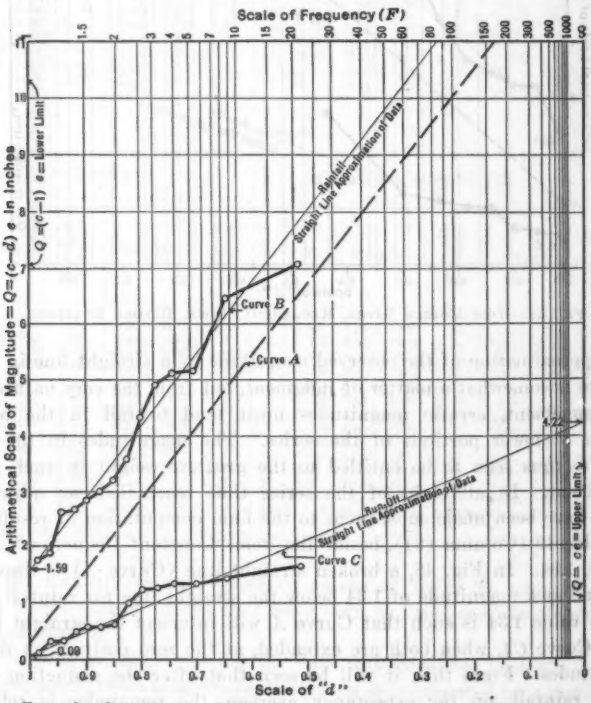

FIG. 49.—DES MOINES RIVER, KEOSAUQUA, IOWA, AFTER DRAINAGE.

Since Curve A is parallel to Curve B (Fig. 48), both curves have the same slope or the same e value in Equation (7). Since the broken straight line, A, intersects zero magnitude at the same position on the d scale as the straight line for run-off, C, both these curves have the same c value. The only difference between Equation (7) as applied to Curve A and to Curve C, is in the respective values of e for rainfall and run-off. The proportion of the remainder of rainfall (after deducting 1.34 in.) appearing as run-off, on the expectancy average for Fig. 48, is then $\frac{3.48}{8.70} = 40$ per cent. These values are given in Column (2) of Table 40.

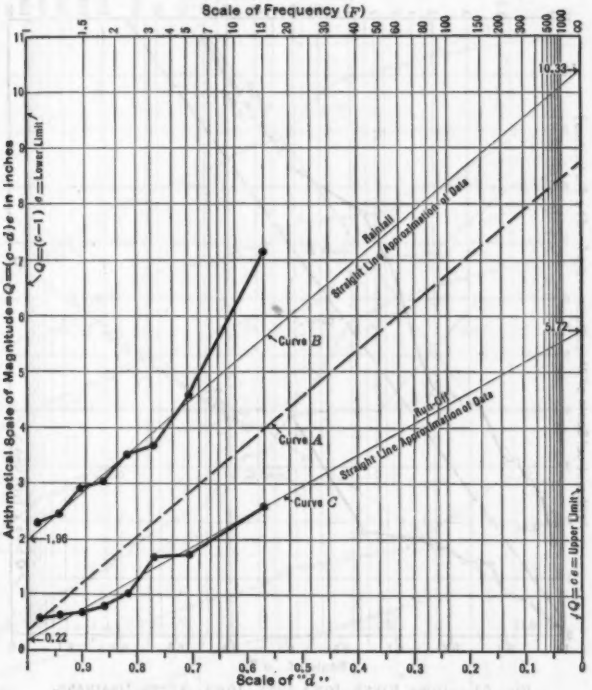

FIG. 50.—IOWA RIVER, IOWA CITY, IOWA, BEFORE DRAINAGE.

A similar process determines the values for the other columns in Table 40, and from these the following relations may be stated:

Des Moines River, Keosauqua, Iowa, Area = 13 900 sq. miles:

Before drainage, the run-off = 40.0% of (rainfall — 1.34 in.)
After drainage, the run-off = 32.6% of (rainfall — 1.31 in.)

Iowa River, Iowa City, Iowa, Area = 3 140 sq. miles:

Before drainage, the run-off = 65.7% of (rainfall — 1.62 in.)
After drainage, the run-off = 47.7% of (rainfall — 1.68 in.)

Assuming that the run-off proportion for the whole area before drainage truly represents the run-off proportion for the undrained area after drainage, then the run-off proportion for the area affected by drainage, on the Des Moines River, is equal to:

$$D = \frac{E - A\,B}{C} = \frac{0.326 - 0.673 \times 0.400}{0.327} = 0.174$$

For the Iowa River the same run-off proportion is equal to,

$$D = \frac{0.477 - 0.535 \times 0.657}{0.465} = 0.271$$

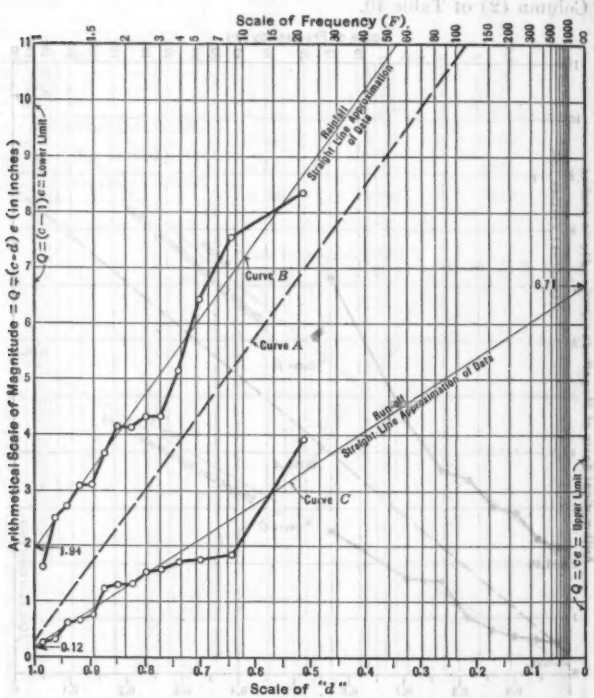

FIG. 51.—IOWA RIVER, IOWA CITY, IOWA, AFTER DRAINAGE.

After the constant deduction, the proportion of run-off from the drained areas on the Des Moines River is reduced from 0.400 to 0.174, or 43.5% of the original proportion. Likewise, the corresponding proportion for the Iowa River is reduced from 0.657 to 0.271, or 41.2% of the original proportion.

Since drainage, for agricultural purposes, is generally installed only in sections having flatter topography and slower rates of flow concentration, the assumption stated as a premise to these final computations is not exactly sound. However, the error thus introduced must be small as compared with the marked effect of drainage which the computations tend to prove.

In the very nature of the regions subjected to drainage improvements, also lies an explanation of the fact that these works show no appreciable effect on the rate of flow depletion after the conclusion of a storm. This rate would seem to be essentially a function of the more flashy portions of the catchment area, and while the writer is compelled to admire the ingenuity of the authors' attack along this line, it is difficult to conceive how drainage works on the slower areas, can be expected to show appreciable effect on this index.

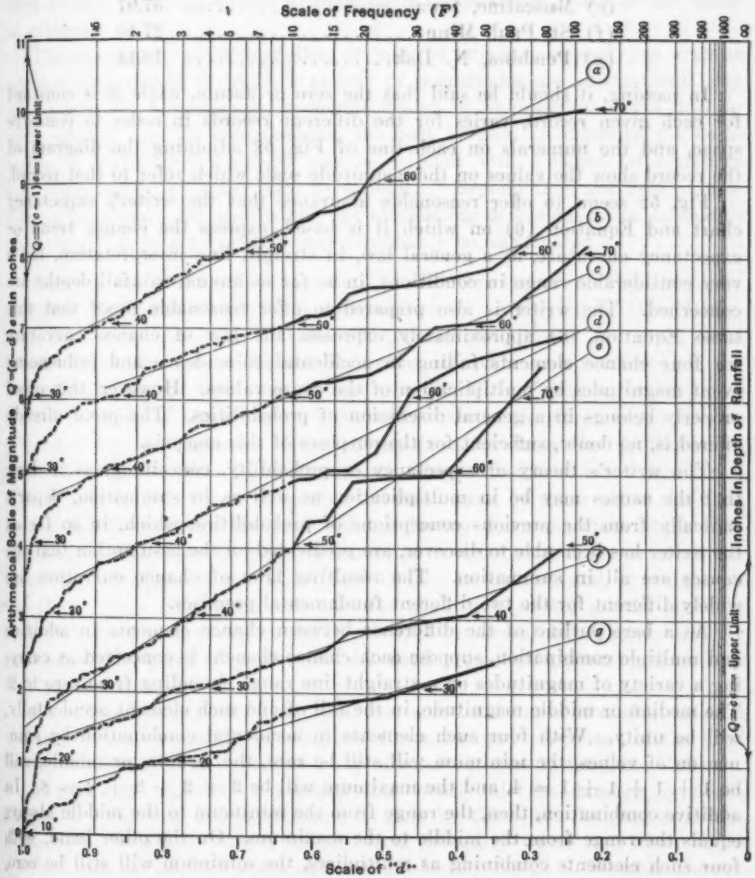

FIG. 52.—ANNUAL RAINFALL DEPTHS.

In Fig. 52, seven of the longer rainfall records of the United States are portrayed on the same chart design used in this analysis. The distribution of the data for each diagram has been effected under the principles of Equation (3), and the straight line approximating the data, in each case, has been

marked by a letter within a circle, for identification. These letters refer to the records published by the U. S. Weather Bureau, listed as follows:

Total annual rainfalls for:	Inches (mean).
(*a*) Boston, Mass.	43.57
(*b*) Philadelphia, Pa.	42.70
(*c*) Marietta, Ohio	41.22
(*d*) St. Louis, Mo.	39.77
(*e*) Muscatine, Iowa	37.07
(*f*) St. Paul, Minn.	27.40
(*g*) Pembina, N. Dak.	18.14

In passing, it should be said that the zero or datum, while it is constant for each given record, varies for the different records in order to conserve space, and the numerals on each line of Fig. 52 adjoining the diagram of the record show the values on the magnitude scale which refer to that record.

Fig. 52 seems to offer reasonable assurance that the writer's expectancy chart and Equation (6) on which it is based, express the chance trend or expectancy curvature, as a general law, in straight line interpretation, for a very considerable range in conditions, in so far as annual rainfall depths are concerned. The writer is also prepared to offer reasonable proof that this same Equation (6) approximately expresses the law of chance curvature for four chance elements falling in accidental coincidence and influencing event magnitudes by multiplication of the cause values. However, this proof properly belongs in a general discussion of probabilities. The proof already offered is, no doubt, sufficient for the purposes of this analysis.

The writer's theory of expectancy or probability, conceiving, as it does, that the causes may be in multiplication as well as in summation, departs radically from the previous conceptions of probabilities, which, in so far as the writer has been able to discover, are predicated on the assumption that the causes are all in summation. The resulting laws of chance curvature are widely different for the two different fundamental premises.

As a bare outline of the difference between chance elements in additive and multiple combination, suppose each chance element is conceived as carrying a variety of magnitudes on a straight-line range extending from zero to 2. The median or middle magnitude, in the fall of one such element accidentally, will be unity. With four such elements in accidental combination by summation of values, the minimum will still be zero, the median, or middle, will be $1 + 1 + 1 + 1 = 4$, and the maximum will be $2 + 2 + 2 + 2 = 8$. In additive combination, then, the range from the minimum to the middle always equals the range from the middle to the maximum. On the other hand, with four such elements combining as multipliers, the minimum will still be zero, the median, or middle, will be $1 \times 1 \times 1 \times 1 = 1$, and the maximum will be $2 \times 2 \times 2 \times 2 = 16$. With four chance elements in multiple combination, it is then evident that the range from median to maximum is fifteen times the range from minimum to median. This may serve to illustrate the radical difference in the curvature under the two conceptions.

It is common knowledge that in the mathematical interpretation of natural phenomena, the known cause elements, especially when rates or velocities enter into the determination, are generally factors in multiplication, instead of items in summation, and it is this fact that tends to support the writer's theory.

The writer hopes that this outlook on the problem under consideration may prove of interest. The subject outlined is endless, and the effort has been to call attention merely to some of the outstanding links in the chain of evidence.

The expectancy method applies, or may be adjusted to apply, to a great number of problems in which a variety of magnitudes are encountered. By this means, for instance, the economic design of engineering works dealing with rainfall and its consequences, the best bargain in capacity for combating or utilizing the run-off may be selected with a degree of certainty that other methods cannot hope to approach. Beyond this, the field of usefulness is perhaps still speculative, but astonishing developments in extending the scope of human knowledge may reasonably be anticipated.

WILLIAM M. HALL,* M. AM. SOC. C. E. (by letter).—This discussion describes the formation of floods and flood crests on the Upper Ohio River, their movement, duration, and relation to the great floods and great flood crests on the Mississippi River. It invites attention to a condition of the movement of storms from west to east over the Ohio Valley which makes it appear probable, if not apparent, that in the case of the Ohio, better service will result by locating some if not all the detention reservoirs on the tributaries entering the Ohio above Cincinnati, Ohio, rather than below Louisville, Ky. It also undertakes to show that the crest wave, or water above the flood line, is the only part of the great volume flowing down the Ohio which it is advisable or necessary to catch in detention reservoirs in order to abate the great flood damage on the Ohio; that it may be caught, and, if caught on the Upper Ohio tributaries and held for about two weeks, it will probably reduce the height of floods on the Mississippi by several feet, thereby greatly increasing the factor of safety for the flood protection project proposed by General Jadwin.

From this Symposium it appears that some additional available data on the subject afford control, showing the formation and movement of the flood crests may be useful in a further study of reservoir sites in the Ohio Valley.

Presumably engineers are aware that nearly all the damage by floods along the Ohio and the Mississippi is done by that part designated as the crest wave, which is formed mainly from tributary water and flows above the flood line or the flood stage plane established by the U. S. Weather Bureau and shown on its daily reports for the gauges at the principal cities and towns along the rivers from Pittsburgh, Pa., to New Orleans, La. To eliminate the damage caused by a flood it is only necessary to reduce or eliminate the crest wave sufficiently; or, in other words, temporarily to detain

* Parkersburg, W. Va.

in reservoirs that volume of water which would flow above the flood stage, as designated by the Weather Bureau.

In general, the floods in the Ohio Valley are produced by storms which approach the Ohio water-shed from the southwest, travel across it from its southerly or westerly boundary to its easterly or northeasterly boundary, a distance of 600 to 800 miles. This meteorologic condition is so uniform it seems safe to assume that, of the great floods, 90% or more occurred from storms which passed from the southwesterly edge of the drainage area to the easterly or northeasterly boundary, occasionally being augmented by a storm from the west or northwest joining the one from the southwest in the area north of the Ohio. Such a condition caused the great flood of 1913. This movement of storms from west to east usually consumes a period of 2 to 4 days from the Mississippi River to the easterly boundary of the Ohio water-shed.

The assembly of a part of the available data on the actual formation and movement of the greatest floods in the Ohio Valley since 1884 appears to explain the formation and movement of flood crests. It may be useful in coming to a final conclusion as to the location of reservoirs in the Ohio Valley for reducing flood crests in the Mississippi, which reservoirs will greatly relieve the Ohio of its present greatest unconquered menace and affliction without undue cost to the Mississippi project.

The writer appreciates the great accomplishment of the authors of the Symposium; also the great amount of work involved in the formulation of a complete report and estimate of cost of the flood control of the Mississippi River in its alluvial valley in so short a time for its presentation to Congress by General Jadwin. Therefore, it would be most surprising if some details of the many elements which affect the problem may not at first have been given too little value. Moreover, as has been well said, it is perfectly sound engineering to consider first the repairs to the levee system, to provide spill-ways or other adequate ways for the release and quick discharge of all the water which may converge from every source in the Lower Mississippi, and thereafter to treat, as an additional factor of safety, the location and building of reservoirs or other aids for the reduction of the floods at Cairo, Ill., and thereby throughout the alluvial valley. In this connection it is well to remember that the flood record of 75 years is very brief for estimating what may occur in the coming 500 to 1 000 years; therefore, it appears that the factor should be made adequate. The writer's work of twenty years on the improvement of the Ohio has involved a familiarity with the formation, movement, and character of its floods and their intimate relation with the Mississippi floods. The additional data here presented may be useful in making the best and possibly the most economical solution of the problem.

In Table 41, the movement from day to day of the crests of thirteen of the largest floods from 1884 to date is shown for nineteen gauge stations, from Pittsburgh to New Orleans. The gauge readings are taken from the records of the U. S. Weather Bureau reports, and from the Mississippi River Commission reports of "highest and lowest annual stages".

The same information is shown graphically in Fig. 53. It also shows the water surface on March 29, 1913, the day of greatest flood stage at Marietta, Ohio, and Parkersburg, W. Va., from which is more easily visualized the effect on that flood of catching the water for five days or more which passed above the flood line from March 27 to April 2 at Catlettsburg, Ky.

An inspection of Table 41 and Fig. 53 in connection with Colonel Kutz' Tables 13 and 14, with a map (Fig. 54) makes it obvious that all or nearly all the great floods in the Mississippi River and the Lower Ohio River have their beginning in the Upper Ohio water-shed. By a comparison of the data it is observed that most of them are not only formed in the Upper Ohio River above the mouth of the Scioto, at Portsmouth, Ohio, but from that locality to Helena, Ark., above the mouth of Arkansas River, the movement down stream is very uniform from all the floods except those of 1897, 1916, and 1920, the movements of which were somewhat checked.

In the flood of 1897 the time of crest at Evansville, Ind.—7 days from Pittsburgh—was prolonged to 28 days in reaching its peak at Cairo, because the very large flood from the Tennessee was unusually slow in arrival at the Ohio. In 1920, the slow movement of 17 days to Cairo appears to be due to a slow and large discharge of several tributaries. In 1916, the 21 days' difference in time of crests at Pittsburgh and Cairo was because the large discharge from the Mississippi (about 41% as large as from the Ohio) was slow in arrival at Cairo. Of the floods tabulated, those of 1916 and 1927 appear to be the only ones in which the Mississippi affected the time of the arrival of the Ohio crest at Cairo, or caused any delay in its movement down stream. The delay was nearly 2 weeks in 1916 and about 2 days in 1927. The quick arrival of the Ohio crest at Cairo in 1907 and 1904 was due to the formation of crests at Pittsburgh on or about the same dates as at Portsmouth, 338 miles farther down stream, which, in turn, was due to unusually large discharges by the tributaries from the Big Sandy to the Kanawhas.

From Table 41, for the thirteen floods the average difference in time of the flood crests at Pittsburgh and Cairo (978 miles) is 13.9 days; omitting the three slow movements in 1920, 1916, and 1897, the average time of passage of crests from Pittsburgh to Cairo is 11.4 days, a minimum of 8 days and a maximum of 16 days. This gives an average velocity of 86 miles per day for the crest movement. From Cincinnati to Cairo, 510 miles, the average difference of time for the thirteen floods is 10 days, or 51 miles per day. From Parkersburg, as far down the Mississippi River as Helena, the movement is fairly uniform, the average time for the 1 285 miles being 23 days, or 47.8 miles per day, the fastest movement being 16 days for the 1907 and 1927 floods, and the slowest, 38 days in 1897. From Helena to New Orleans, 657 miles, the average difference in time of passage of the flood crests given as 21 days is obtained without including the 1927 flood, when an early crest was produced by crevasses and an artificial by-pass which effectually stopped the rise at New Orleans 2 days before the head rise reached Helena.

From these data, personal observation, and study of flood movement, the writer believes that in great rivers like the Ohio, between banks from $\frac{1}{4}$

TABLE 41.—TIME OF PASSAGE OF CRESTS OF GREATEST FLOODS IN OHIO RIVER FROM SOURCE TO MOUTH, AND ON MISSISSIPPI RIVER FROM CAIRO, ILL., TO NEW ORLEANS, LA.

Name.	Distance, from Pittsburgh, in miles.	Flood stage, in feet.	Date of Crest of Flood. 1927.		1922.		1920.		1917.		1916.		1913.		1912.	
			Date.	Stage.	Date.	Stage.	Date.	Stage.	Date.	Stage.	Date.	Stage.	Date.	Stage.	Date.	Stage.
Pittsburgh, Pa.	0	25	4/7	16.8	3/16	18.3	3/14	24.5	3/13	23.0	1/13	17.1	3/28	30.4	3/31	17.5
Beaver, Pa.	29	30	4/8	15.8	3/16	23.7	3/14	36.3	3/14	32.2	1/14	25.5	3/28	46.3	3/31	26.7
Wheeling, W. Va.	95	43	4/8	17.3	3/17	28.2	3/15	38.5	....	....	1/14	29.6	3/28	51.1	4/1	27.0
Marietta, Ohio.	171	33	....	....	3/16	27.0	3/15	34.4	....	....	1/15	29.2	....	....	....	....
Parkersburg, W. Va.	184	36	4/10	18.3	3/16	30.0	3/16	34.9	3/15	38.4	1/15	30.8	3/29	58.9	4/1	29.2
Point Pleasant, Ohio.	265	40	4/11	28.7	3/17	39.3	3/17	40.4	3/16	47.1	1/15	37.5	3/30	62.8	4/5	40.8
Catlettsburg, Ky.	319	50	4/11	37.5	3/17	49.0	3/21	53.5	....	....	1/15	45.7	3/31	67.7	4/4	49.2
Portsmouth, Ohio.	355	50	4/12	37.7	3/18	49.9	3/21	52.9	3/17	54.4	1/15	49.3	3/31	67.9	4/5	49.8
Cincinnati, Ohio.	468	52	4/13	39.3	3/18	52.2	3/22	54.6	3/17	56.1	1/15	53.1	4/1	69.8	4/6	51.7
Louisville, Ky.	602	8	4/14	16.9	3/19	30.2	3/23	31.5	3/19	30.5	1/15	31.2	4/2	44.9	4/7	28.5
Evansville, Ind.	792	35	4/16	36.3	3/21	42.9	3/25	42.8	3/22	42.9	1/18	43.6	4/5	48.4	4/9	42.5
Paducah, Ky.	934	43	4/18	47.2	3/24	48.8	3/28	45.3	3/25	47.1	1/19	45.7	4/7	54.3	4/8	49.9
Cairo, Ill.	978	45	4/20	56.4	3/26	58.6	3/31	51.4	4/4	50.1	2/4	53.4	4/7	54.8	4/6	54.0
Memphis, Tenn.	1 205	35	4/22	45.9	4/1	42.5	4/5	40.3	4/10	40.4	2/9	43.4	4/9	46.5	4/6	45.2
Helena, Ark.	1 285	44	4/26	56.7	4/3	58.1	4/8	50.1	4/12	49.9	2/11	53.4	4/22	55.2	4/22	54.3
Vicksburg, Miss.	1 580	45	5/4	58.7	4/27	54.8	4/27	50.9	4/23	50.0	2/15	53.8	4/27	52.3	4/12	51.6
Natchez, Miss.	1 683	46	....	....	4/26	55.3	4/28	51.5	4/25	49.9	2/15	53.5	4/27	52.6	4/14	51.4
Baton Rouge, La.	1 819	35	....	....	5/16	45.7	5/22	41.5	4/29	37.8	3/1	42.6	5/9	41.3	5/11	43.8
New Orleans, La.	1 942	17	4/24	21.0	4/25	21.3	5/18	19.2	5/2	16.9	3/3	20.1	5/5	20.3	5/11	21.1
			Crest Movement, Number of Days.													
Pittsburgh, Pa., to Cairo, Ill.	....	....	13		10		17		12		21		10		10	
Cincinnati, Ohio, to Cairo, Ill.	....	....	7		8		9		8		19		7		3	
Parkersburg, W. Va., to Helena, Ark.	....	....	16		18		23		28		26		22		21	
Helena, Ark., to New Orleans, La.	....	....	— 2		22		40		20		21		13		19	

TABLE 41.—*(Continued.)*

Name.	Distance, from Pittsburgh, in miles.	Flood stage, in feet.	Date of Crest of Flood.											
			1907.		1904.		1903.		1898.		1897.		1884.	
			Date.	Stage.	Date.	Stage.	Date.	Stage.	Date.	Stage.	Date.	Stage.	Date.	Stage.
Pittsburgh, Pa.	0	25	1/20	23.2	3/28	15.2	3/1	28.9	3/24	28.5	2/24	28.9	2/6	31.9
Beaver, Pa.	29	30	1/20	33.7	3/28	23.6	3/1	38.1	3/24	42.3	2/24	37.7	....	....
Wheeling, W. Va.	95	43	1/20	36.1	3/28	24.2	3/2	40.2	3/24	45.9	2/25	37.0	2/8	46.5
Marietta, Ohio	171	33	....	....	....	....	....	....	....	....	2/25	36.0	2/9	52.0
Parkersburg, W. Va.	184	36	1/21	39.9	3/29	26.6	3/3	39.4	3/26	47.8	2/25	37.5	2/9	55.
Point Pleasant, Ohio	265	40	1/19	52.0	3/29	30.3	3/4	45.0	3/27	51.2	2/25	52.3	....	....
Catlettsburg, Ky.	319	50	1/20	59.9	3/28	36.4	3/4	49.7	3/28	56.0	2/25	58.6	....	....
Portsmouth, Ohio	355	50	1/20	60.9	3/29	39.3	3/4	51.8	3/28	57.3	2/25	59.0	....	....
Cincinnati, Ohio	468	52	1/21	65.1	3/27	45.7	3/5	58.2	3/29	61.4	2/26	61.1	2/14	71.1
Louisville, Ky.	602	28	1/22	41.4	3/28	22.9	3/9	28.5	3/30	36.3	2/28	35.4	2/16	46.6
Evansville, Ind.	792	35	1/24	46.2	4/3	39.8	3/11	42.4	4/2	44.8	3/2	43.6	2/18	48.8
Paducah, Ky.	934	43	1/28	45.7	4/4	44.7	3/15	47.6	4/6	47.3	3/24	50.9	2/22	54.2
Cairo, Ill.	978	45	1/28	50.4	4/5	49.1	3/15	50.6	4/6	49.8	3/25	51.6	2/22	51.8
Memphis, Tenn.	1 205	35	2/3	40.3	4/11	39.2	3/20	40.1	4/10	37.3	3/20	37.6	3/1	34.2
Helena, Ark.	1 285	44	2/6	50.4	4/15	47.6	3/26	51.0	4/17	49.1	4/4	51.7	3/6	47.0
Vicksburg, Miss.	1 580	45	2/12	49.6	4/25	46.8	3/28	51.8	4/23	49.4	4/16	52.5	3/25	49.0
Natchez, Miss.	1 683	46	2/13	48.9	4/25	45.5	3/28	50.3	4/26	47.4	5/2	49.8	3/26	47.4
Baton Rouge, La.	1 819	35	2/17	37.4	4/29	33.7	4/7	40.1	4/29	34.6	5/13	40.6	3/24	36.2
New Orleans, La.	1 942	17	2/16	18.6	5/1	16.1	4/6	19.4	5/1	17.0	5/13	19.5	3/18	15.6
			Crest Movement, Number of Days.											
Pittsburgh, Pa., to Cairo, Ill.	....	....	8		8		14		13		28		16	
Cincinnati, Ohio, to Cairo, Ill.	....	....	7		9		10		7		27		8	
Parkersburg, W. Va., to Helena, Ark.	....	....	16		17		23		22		38		25	
Helena, Ark., to New Orleans, La.	....	....	10		16		11		16		39		12	

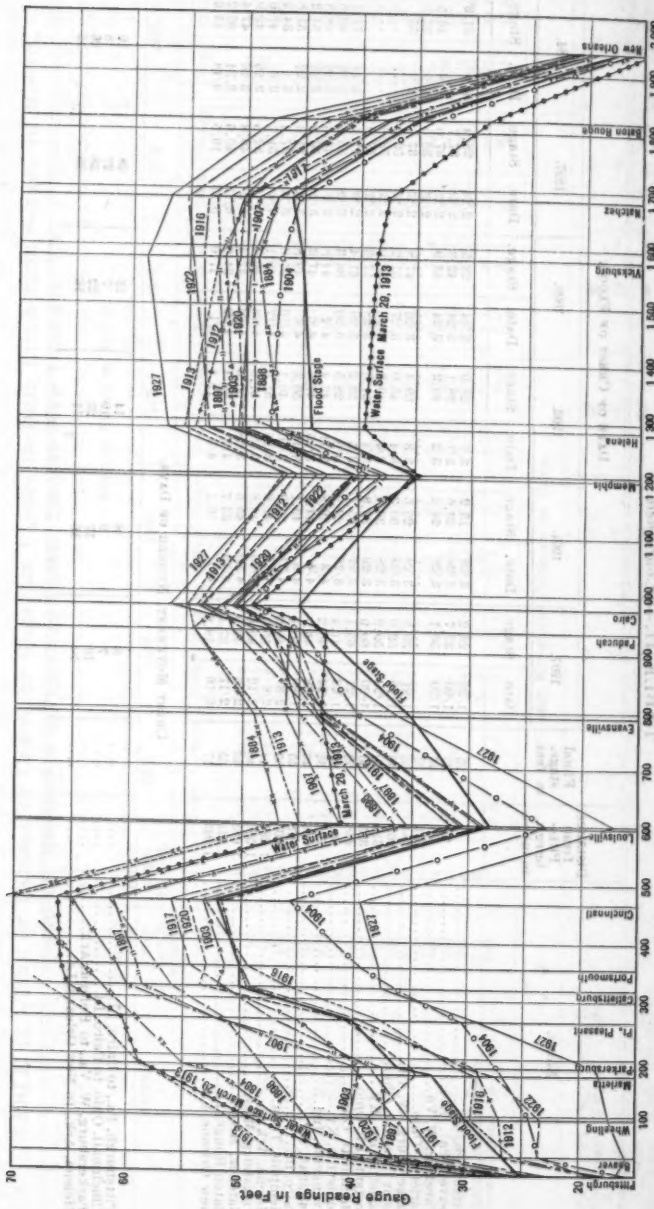

FIG. 53.—FLOOD STAGES OF THE OHIO AND MISSISSIPPI RIVERS.

mile wide at Parkersburg to 1 mile wide near Cairo, and the Mississippi, from ½ to 1½ miles wide between Cairo and the mouth, the great mass of water, flowing 20 ft. or more above the bed of the river and 300 ft. or more from the shores, is so little affected by the friction of bed and banks that the velocity is nearly uniform throughout any section, except where it is more or less disturbed by the inflow from large tributaries and by going over the 26-ft. fall at Louisville.

As may be observed from Fig. 54, between the Scioto, at Portsmouth (above which the governing crest peak is usually formed) and Cairo, 610 miles, there are only seven tributaries large enough to cause much disturbance in the crest wave; or, including the Falls, only one dislocation of relative position of the particles of water in the crest wave for each 75 miles of river. It is believed, therefore, that in any great flood crest passing Portsmouth, if the particles of water in any section of the crest, 100, 200, or 300 miles long, could be located and their relative position determined, when passing Cairo, it would be found that in the corresponding section nearly all the water in crest near Portsmouth would be found near Cairo, of course, augmented by the intermingling of the increments of inflow from tributaries.

In a discussion of this subject* the late James A. Seddon, M. Am. Soc. C. E., designated such condition as "perfect flow". The crest wave of approximately so-called perfect flow is probably about 1 000 ft. wide in the section of river from Catlettsburg, at the mouth of the Big Sandy, to Cincinnati, and about 2 200 ft. wide below Paducah, at the mouth of the Tennessee. Due to this increase of width and also to an increase of several miles in width of back-water into side streams and overflowed bottom-land from Paducah to Cairo and down the Mississippi, the effect of catching any definite part of the crest-wave water in the upper section of the river cannot be estimated with accuracy at Cairo.

Although topographical maps of the area up to the high-water plane are not available, it is known that the flood-plane broadens out to a width of several miles over which this crest wave must spread at stages above 45 to 50 ft.; and at its entrance to the Mississippi, it must flow in between levees about 1¼ miles apart. With such approximate data, the water surface area for that 100 miles of river, or the acre-feet of water per foot of depth, is estimated to be approximately 136 000. It appears, therefore, that should the daily discharge of the crest wave from above Cincinnati be reduced by 300 000 to 400 000 acre-ft. (or 150 000 to 200 000 cu. ft. per sec.) for a period of from 3 to 10 days, while the peak of the crest is passing Cairo and entering the Mississippi, the crest height would certainly be reduced by 2½ to 4 or 5 ft., the exact amount depending upon local conditions, such as the stage of the crest peak, the discharge of the Mississippi River above the Ohio, the impounding effect of the levees, etc. Therefore, observations of flow and comparisons of discharge at respective localities appear to be the best guide, until more exact information is obtained as to areas flooded at high stages. The reader may more readily visualize the effect simply by inspecting Fig. 53 and imagin-

* Annual Report, 1892, U. S. Chf. of Engrs., p. 2907.

FIG. 53.—FLOOD STAGES OF THE OHIO AND MISSISSIPPI RIVERS.

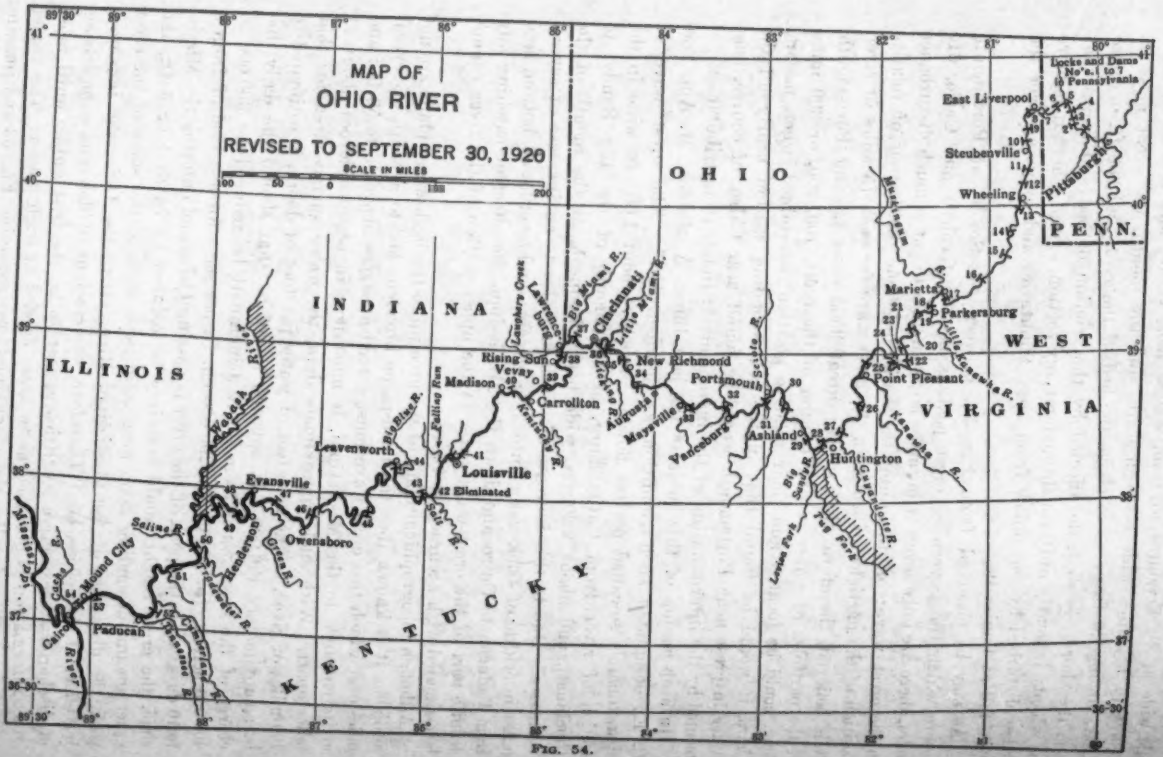
MAP OF
OHIO RIVER
REVISED TO SEPTEMBER 30, 1920
SCALE IN MILES
100 50 0 100 200
OHIO
INDIANA
ILLINOIS
KENTUCKY
WEST VIRGINIA
PENN.
Locks and Dams Nos. 1 to 7 in Pennsylvania
East Liverpool
Steubenville
Wheeling
Pittsburgh
Marietta
Parkersburg
Point Pleasant
Huntington
Ashland
Portsmouth
New Richmond
Cincinnati
Little Miami R.
Big Miami R.
Lawrenceburg
Aurora
Rising Sun
Vevay
Madison
Carrollton
Kentucky R.
Augusta
Maysville
Vanceburg
Big Sandy R.
Guyandotte R.
Kanawha R.
Monongahela R.
Muskingum R.
Hocking R.
Scioto R.
Big Blue R.
Falling Run
Leavenworth
Louisville
42 Eliminated
Salt R.
Evansville
Owensboro
Henderson
Green R.
Tradewater R.
Saline R.
Mound City
Cairo
Mississippi R.
Ohio R.
Paducah
Cumberland R.
Tennessee R.
Wabash R.
Lock and Dam
Canal

FIG. 54.

ing all the water withheld from Beaver, Pa., to Louisville enclosed by the flood-stage line and the various crest lines.

A study of the greatest floods which passed Cincinnati, 468 miles below Pittsburgh, from 1870 to 1921 (Fig. 55), shows that there were thirty-eight up to or above the flood line (52 ft.) and that their duration above that line was from 1 day to 18 days; that six of them were from 1 day to 10 days above the 62-ft. stage (10 ft. above flood stage), the total being only 30 days in 51 years. At Cincinnati, the fluctuation between high and low water is the greatest on the Ohio, and the duration of floods is probably as great or greater than at any other locality above Paducah at the mouth of the Tennessee.

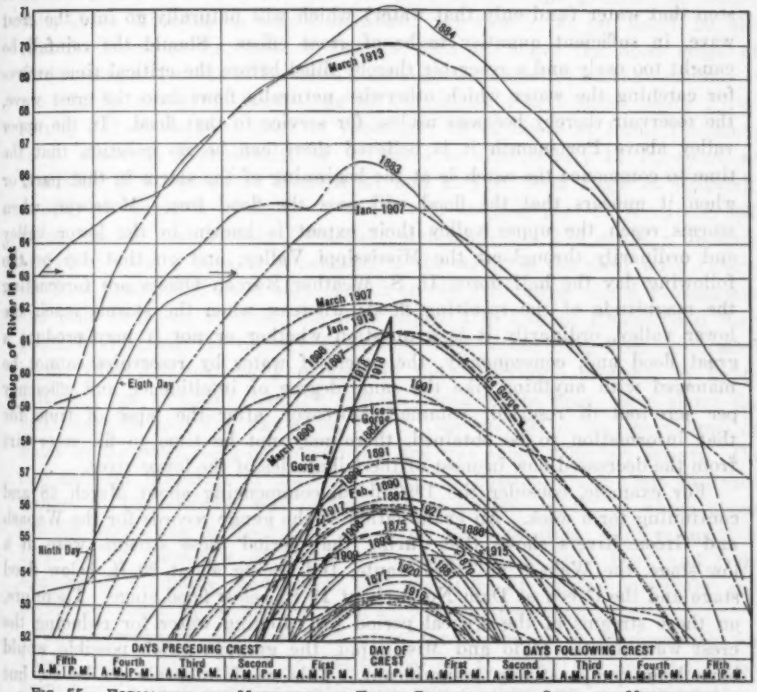

FIG. 55.—FORMATION AND MOVEMENT OF FLOOD CRESTS IN THE OHIO AND MISSISSIPPI RIVERS, SHOWN BY HYDROGRAPHS AT CINCINNATI, OHIO.

Thus, the crest waves, which do practically all the damage along the Ohio, are generally of short duration. As compared with the great volume of water which goes down the river, it is evident that only a small fraction of 1% needs to be caught in order to eliminate the damage; and with one exception in 58 years it has passed in the four months, January to April, inclusive. By a similar study, the passage of the largest floods by Cairo is found to range with one or two exceptions from 12 to 31 days. Therefore, it

is probable that the water which passes Cairo before the stage has reached the flood line, that is, prior to 5 to 10 days before the passage of the peak of the crest, ordinarily does no damage. It certainly does no damage unless it happens to meet a discharge from one or more of the largest tributaries down stream sufficient to produce a flood stage in the lower river; or, unless bottled up by insufficient outlets, a condition which will be relieved by General Jadwin's plan.

Due to this formation and movement of flood crests from the upper river and the condition of ordinarily having rain storms in the Lower Ohio Valley several days prior to those in the upper valley, reservoirs built in the lower valley will probably be found more difficult to fill at just the right time to stop that water (and only that water) which will naturally go into the crest wave, in sufficient quantity to be of great effect. Should the rainfall be caught too early and a reservoir thereby filled before the critical time arrives for catching the water which otherwise naturally flows into the crest wave, the reservoir thereby becomes useless for service to that flood. In the upper valley above Portsmouth it is believed there can be no question that the time to commence the catch is at the beginning of the storm in that part, or when it appears that the flood will pass the flood line. Moreover, when storms reach the upper valley their extent is known in the lower valley and ordinarily throughout the Mississippi Valley, and on that day or the following day the half dozen U. S. Weather Bureau Offices are forecasting the magnitude of the resulting flood; whereas, when the storms reach the lower valley, ordinarily it is a question whether or not it may produce a great flood and, consequently, the catch of water by reservoirs cannot be managed with anything like the same degree of intelligence and efficiency per acre-foot of reservoir volume. Moreover, after the lapse of time for that information to be obtained, there may not be time to fill reservoirs from the decreased flow in most of the tributaries of the lower river.

For example, consider the 1920 flood, commencing about March 28 and continuing for a week. An examination of the gauge records for the Wabash and Green Rivers shows that during that period these streams were at a low stage, the Wabash at Terre Haute, Ind., being about 20 ft. below flood stage and the Green at Dam No. 4 about 18 ft. below flood stage. Therefore, on those streams at the critical period for catching water for reducing the crest wave in the Ohio and Mississippi, the greatest catch possible would have been comparatively insignificant. This would not always be so, but it appears that it would frequently be so. The data on those streams for the earlier floods are not available. It is not claimed that a reservoir on the Wabash or on the Cumberland or on the Tennessee would not be desirable or effectual for some floods or be inadvisable to build, but it is believed the condition named will result in more difficulty in their efficient operation, or in less efficiency than for those in the upper river. Regularity of the movement of the crests from Pittsburgh or above Portsmouth to Cairo largely reduces the objection to the greater distance from Cairo of detention reservoirs on the upper tributaries between the Scioto and the Beaver.

Moreover, should it be determined to operate reservoirs on the lower river solely for the benefit of the Mississippi, that is, to let the crest wave in the tributary run out without restraint regardless of its local damage, and thereafter catch the water which would naturally go into the Ohio and Mississippi River crest wave, a serious conflict of interest might result. It is for this reason that it has been concluded that the Miami River Conservancy cannot be operated for the protection of the Miami Valley and at the same time for the protection of the Mississippi River.

Data for accurately determining the quantity of water in the crest wave are not available, therefore, only an approximate estimate is possible.

The greatest flood of record at Pittsburgh was in 1907 and the greatest on the Mississippi, in 1927, and the 1913 flood is the greatest of record from above Wheeling to near Cincinnati. At 6:00 P. M., on March 29, the 1913 flood was at its crest of 58.3 ft., 22.3 ft. above the flood stage at Parkersburg and about 25 ft. above flood stage at Marietta; on the same day it was slightly below the flood stage at Coraopolis, Pa., 9 miles below Pittsburgh; 7 ft. above flood stage at Wheeling; 20 ft. above at Point Pleasant; 16 ft. above at Portsmouth; 14 ft. above at Cincinnati; 13 ft. above at Louisville; 8 ft. above at Evansville; and at just flood stage at Paducah, possibly the greatest quantity of water ever in that part of the river, 925 miles. In the part between Coraopolis and Louisville, 593 miles, a distance of flow of about 5 days, it is believed, if the water in the crest wave (above the flood line) could have been caught on the tributaries above Portsmouth and held for 15 or 20 days, the time required to flow to Helena, and then gradually released, the damage from that flood would have been eliminated in the Ohio and materially reduced in the Mississippi, probably by 2½ to 4 ft. in stage at Cairo. As the average depth of the crest wave above the flood stage on that day was about 12½ ft., for the 593 miles, the volume of water in the crest wave or above flood line was about 2 000 000 acre-ft. On that date the discharge of the Ohio under the Baltimore and Ohio Railroad Bridge, at Parkersburg, was probably about 700 000 sec-ft., of which about 300 000 sec-ft. was in the crest wave above the flood line. This estimate is based on an inspection of the river, by traversing it in a boat, and from memory of discharge observations and velocities at various low-water and high-water stages up to 45 ft. at Dam 18, 5 miles up stream.

An independent measurement* gives the discharge at Parkersburg for 7 days, March 26 to 31, as 323 654 400 000 cu. ft. (7 450 000 acre-ft.) which harmonizes with the writer's estimated rate of discharge. Applying those rates for the 5 days the river was above the flood stage at Parkersburg and Marietta, it is found that about 1 490 000 acre-ft. passed that locality flowing above the flood-plane and 5 960 000 acre-ft. flowing below the flood-plane; or, if 1 500 000 acre-ft. of that water had been caught at the proper time from the Little Kanawha, Muskingum, Beaver, or other tributaries, and held for 10 days or more before releasing, there would have been no flood damage in that part of the Ohio; and, thereby, a corresponding reduc-

* *Bulletin Z*, U. S. Weather Bureau, 1913.

tion would have occurred in the crest height to the mouth of the Ohio and in the Mississippi until the crest reached the water impounded by the action of the levees. The discharge of the same flood wave at Cincinnati (Mile 470) above the flood-plane, was a little more than double the corresponding discharge at Parkersburg, that is, somewhat in excess of 3 000 000 acre-ft. It was 11 days in passing, and its peak was 17.9 ft. above the flood line. Had that 3 000 000 acre-ft. been caught and held for 15 to 20 days, it would at least have eliminated most of the damage above Paducah and beyond reasonable doubt would have reduced the crest by several feet from Cairo to Helena, Ark., or farther down, if it had not become impounded by levees.

Should a great flood in the Mississippi be mainly produced from other tributaries as occurred in the exceptional case of the 1927 flood, a detention reservoir system in the Upper Ohio Valley could be made effective in reducing stages in the Mississippi by closing the detention dams when that condition was observed from the rainfall or from gauge readings of its other tributaries. It would be entirely practicable to fill the reservoirs, provided the flow in the Ohio was not reduced to a stage below a 10-ft. navigable stage, a very improbable condition. For example, when the flood of 1927 was forming, it was known 10 days before the crest peak passed Cairo that a great flood was approaching, and 5 days before, it was remarked in the Engineer Office at Louisville that the previous record at Cairo would probably be broken.

As the Upper Ohio Valley along the banks of the Ohio River is much more thickly populated than either the part below Louisville, or the Mississippi Valley, and as above Parkersburg the greater part of its banks are densely occupied by industrial plants, the money loss by floods is probably greater per mile of river than from Louisville to New Orleans. The money damage to Dayton and the Miami Valley by the same flood has been stated to be from $30 000 000 to almost $100 000 000. By the same measure for estimating, the loss in the Ohio Valley above Cincinnati by the same flood must have been much more.

It should be realized that on the way down these two great rivers, 1 942 miles from Pittsburgh to New Orleans, it is the same crest water that passes Pittsburgh, Marietta, Parkersburg, Cincinnati, Louisville, and Cairo, doing such vast damage in its passage; and it is a large factor, usually the principal factor, in the formation of the crest wave at Cairo (its contribution in the thirteen floods tabulated being from 51 to 87% of the water passing Cairo at crest stages), which proceeds on down the Mississippi to New Orleans, producing many more millions of damage and untold human suffering. Now seems to be the time for the National Government to join with the Ohio Valley States in eliminating this great menace to property, life, and happiness, by managing it as a part of the larger Mississippi River project.

To those familiar with the construction and operation of a series of locks and movable dams for navigation, such as that on the Ohio, the problem of a system of detention reservoirs appears little or no more difficult; certainly not so difficult as the proposed Ohio System appeared twenty-five years ago. No one can read the Weather Bureau meteorologic report of the rainfall which produced the 1913 flood without concluding that

a greater flood is a possibility if not a probability. That condition, combined with the fact that population and property values are continually and rapidly increasing, makes it appear that the construction of detention reservoirs is only a matter of time. If so, it is possible that land values required for the reservoir sites may increase more rapidly than the interest on the money required for construction.

An estimate of cost of catching 2 000 000 or 3 000 000 acre-ft. of water from any one of the major floods noted is an engineering problem of no greater difficulty than many others which have been successfully solved by many engineers employed by the Government and by many other members of the Society. Of course, such provision for future floods is more difficult, but from a study of the records of all recent great floods over the entire valley, it appears that a system of reservoirs may be devised that will completely or materially reduce the destructive part of every crest wave, and reduce its height at Cairo by several feet; provided the reservoirs are operated so as to be filled from the water which would otherwise go into the crest wave.

It is a customary practice to forecast for the dams under construction on the Ohio, the probable rise from an up-river crest 100 to 500 or 600 miles distant. In the accuracy of such forecast is frequently involved the flooding of coffer-dams, the suspension of construction, and the lay-off from work of hundreds of workmen. From the charts used for such forecast prepared from the record of actual floods the following data are abstracted: Storms producing a flood crest at Catlettsburg, Mile 319, of 31.4 ft., on September 9, 1926, produced rises, as follows:

At Dam 46, Mile 757, about 4½ days later, 8 ft. to a stage of 21 ft.
At Dam 52, Mile 939, about 7 days later, 7 ft. to a stage of 22.5 ft.
At Dam 53, Mile 963, about 7½ days later, 7 ft. to a stage of 29 ft.

Data from this and similar floods were plotted for use as guides in forecasting because the conditions of discharge from tributaries during the periods of their flow appeared to be near the normal. It is realized that the rises indicated are not entirely due to the rise at Catlettsburg, but evidently they are largely due to it. Hence, if all the water which produced the rises at Catlettsburg had been caught and held for a period of 5 to 8 days and then gradually released, no rise therefrom could have occurred at Dam 53, 16 miles above Cairo, where gauge readings fluctuate nearly alike by changes in discharge from above Cairo on either river.

There are no data for determining with exactness the reduction in rise at Cairo from such a catch above Catlettsburg, because no two floods on the Ohio are exactly alike, and because of tributaries the discharge from each of which is a factor in the problem. Notwithstanding, it is customary practice to make such forecasts for periods of several days and distances of several hundred miles and to name within a few hours the arrival of the peak of the crest and its gauge reading within a few tenths of a foot. On the accuracy of such forecasts is based action of much moment.

A further reason for locating a part of the proposed reservoirs on the upper instead of on the lower tributaries of the Ohio is that they may also be made to serve as storage dams for filling the pools of movable dams immediately after raising them. As is well known to navigators on the Ohio, for several days after raising the movable dams there is usually a deficiency of depth of water in the upper part of pools for navigation. The defect has not yet been corrected. When all the locks and dams are finished in 1929, traffic may demand a remedy. It may then be found that to store water on some of the tributaries is the only cure for this navigation defect. As such shortage of water only occurs after the flood season, flood-control dams and navigation-control dams would readily serve the dual purpose.

As to finding possible locations for dams and reservoirs, the tributaries above the Scioto are hundreds of miles long, and in a hilly and mountainous country, the question of engineering may be found to be largely that of comparative costs between the best locations. For example, take the possibility of a choice being made of the Little Kanawha River Valley, the tributary entering at Parkersburg (Mile 185). Between points about 32 and about 102 miles above Parkersburg, there are no railways, no paved highways, a sparse population, no factories of great value, only one bridge, and with a difference in elevation of water level for the 70 miles of about 100 ft. With one or two dams in that valley it is probable that 20 or 25% of the desired storage may be provided, or a storage sufficient for catching the entire discharge for a period of 6 to 10 days. As the writer has traversed the valley he feels safe in stating it is almost beyond question that it will be possible to found such dams on rock ledge at or near the river-bed level.

In conclusion, the reason for building storage reservoirs in the upper valley of the Ohio in preference to the lower valley, thus increasing the factor of safety in the Mississippi River flood-control project, may be briefly stated as follows:

(*a*) The catch of the water in the Upper Ohio tributaries can probably be made with more precision as to efficiency in reducing the crest of Mississippi River floods than one in the lower tributaries.

(*b*) It will add 600 miles or more of river flood protection in the most thickly populated part of the entire Ohio water-shed.

(*c*) The storage dams for flood protection may be used, without detriment thereto, during the low-water season for navigation water supply, thereby improving navigation depths in the pools of the movable dams more quickly than can be done otherwise.

(*d*) Surveys in the Upper Ohio Valley may disclose as favorable and economical sites per acre-foot of storage as in the lower valley.

(*e*) As there is greater population and greater damage in the upper river, there is correspondingly greater wealth of property to be damaged, and, therefore, local co-operation may be more readily obtained and organized. It probably only needs the co-operation of the Government organizing force with

the Ohio River States to induce them to join in planning for the work of construction.

(*f*) Reservoirs in the upper valley will be of as much, or nearly as much, protection to the Lower Ohio River communities along the river as if they were on the tributaries in the lower valley; and their operation should be made as much as 90% efficient in catch of the actual discharge of flood crest water from each storm.

(*g*) Finally, the reduction of 2, 3, or 4 ft. in flood crests at Cairo and thence on down the Mississippi to New Orleans, by detention reservoirs in the Upper Ohio Valley, may be found to be the most positive, the most economical, and the most reliable way of producing such result for the 1 900 miles of river within those limits of great floods. For the welfare of the millions of people dwelling or employed along the Ohio and Mississippi Rivers, it is a question of as great importance as any which ever arose not involving their liberty. Now is the opportune time for its consideration by all those most concerned.

C. E. Ramser,* M. Am. Soc. C. E. (by letter).—The subject of the excellent paper by Professors Woodward and Nagler has long been the cause of a great difference of opinion among drainage engineers, based largely on personal observation, often local in character. The results presented in their paper are especially valuable since they are based on actual measurements of run-off made both before and after extensive drainage operations on the water-shed.

The writer's remarks will be confined solely to the rate of run-off or peak run-off. It is believed that the difference of opinion mentioned has been due chiefly to the fact that the rate of run-off is increased in some instances and reduced in others, depending on the nature of the drainage, governing conditions of the water-shed, such as soil and covering; and intensities and amounts of rainfall considered. As a general rule open-ditch drainage is regarded as increasing the rate of run-off, and tile drainage as decreasing it. This statement being true, it is readily apparent that combinations of the two types of drainage tend to complicate the problem greatly, since, with a certain combination of the two types, drainage would produce no effect on the rate of run-off.

In general, it may be said that the effect of both open-ditch and tile drainage on the rate of run-off is primarily governed by changes in the time of concentration for the water-shed and storage conditions. The maximum rate of run-off for a given rainfall intensity occurs when water from every part of the water-shed reaches the channel at the lower end. The rain that produces this maximum rate must continue as long as the time required for water to flow from the most remote point to the lower end of the water-shed, and the greater the intensity of this rain the greater will be the rate of run-off. It is a well-established fact that, in a general way, the intensity of rainfall varies inversely as the duration. From the foregoing it follows that the reduction in the time of concentration of a water-shed increases the rate of run-off. Open ditches which increase the

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velocity of the water reduce the time of concentration for the water-shed. Tile or sub-drainage tends to increase the time of concentration since a longer period is generally required for the water to move through the soil and sub-drainage channels than over the ground surface and natural channels. Where open ditches induce sub-drainage for long distances through pasty or sandy soils, the effect of sub-drainage would be to counteract the effect of open-ditch drainage on the rate of run-off.

The storage of water on a water-shed tends to reduce the rate of run-off. Open ditches which eliminate storage in swamps and old lake beds, or reduce the height and duration of overflows on adjoining lands, have the effect of increasing the rate of run-off for rains such as are ordinarily provided for in drainage improvements. It is conceivable that a rain could occur, with a duration greatly in excess of the time of concentration of the water-shed, that would produce practically the same maximum rate of run-off before and after drainage for certain water-sheds. In tile or sub-drainage the storage of water in the soil tends to reduce the rate of run-off to the point of saturation of the soil.

It appears to the writer that while the conclusions reached by Professors Woodward and Nagler are sound for water-sheds on which both tile and open-ditch drainage have been extensively practiced, it is not believed that the results are generally applicable to all water-sheds in the Mississippi Valley, such as the St. Francis and Little Rivers on which considerable open-ditch drainage, but comparatively little tile drainage, has been done.

T. KENNARD THOMSON,* M. AM. SOC. C. E. (by letter).—It has been estimated that the flood of 1927 caused a loss of $1 000 000 000, of more than 200 lives outright, and that more than 700 000 people lost their homes. In addition, the number of people whose health was shattered and whose lives were shortened from exposure and worry, must be appalling.

While the 1927 flood was the highest, those of 1912 to 1916 and many other years were also very high. Even the so-called ordinary floods, of frequent occurrence, cause enormous direct and indirect losses. Surely, therefore, a radical remedy is justifiable.

Levees which raise the water level 50 ft. above the surrounding country are a standing menace to life and property, especially as they are so easily destroyed.

The writer has read with great interest and some sorrow the various papers of the Flood Symposium. None of the very able engineers has taken the "bull by the horns" and submitted a real remedy.

The proper solution is the application of the old homely saying that, "if one horse cannot pull a load get two or more". The specific plan devised by the writer is shown in Fig. 56. He is satisfied that the Mississippi River problem can be economically solved by creating four new rivers, with non-corrodible banks and bottoms, one in the approximate location of the present river, and the other three as Rivers *A*, *B*, and *C*. For the sake of clearness

* Cons. Engr., New York, N. Y.

only a few contours are shown and the thousands of smaller rivers and streams have been omitted.

River A.—The construction of River *A* should start at the Gulf of Mexico, about 300 miles, more or less, west of the mouth of the Mississippi River; 100 miles from its mouth, it would have an elevation of 100 ft.; 300 miles, 500 ft.; and 700 miles, 700 ft. It would tap the Missouri River near Kansas City, Mo., at a cost of less than $200 000 000 per year for 20 years; but long before reaching the Missouri the profits from the construction should be realized, thus ensuring the construction of Rivers *B* and *C* and the reconstruction of the Mississippi.

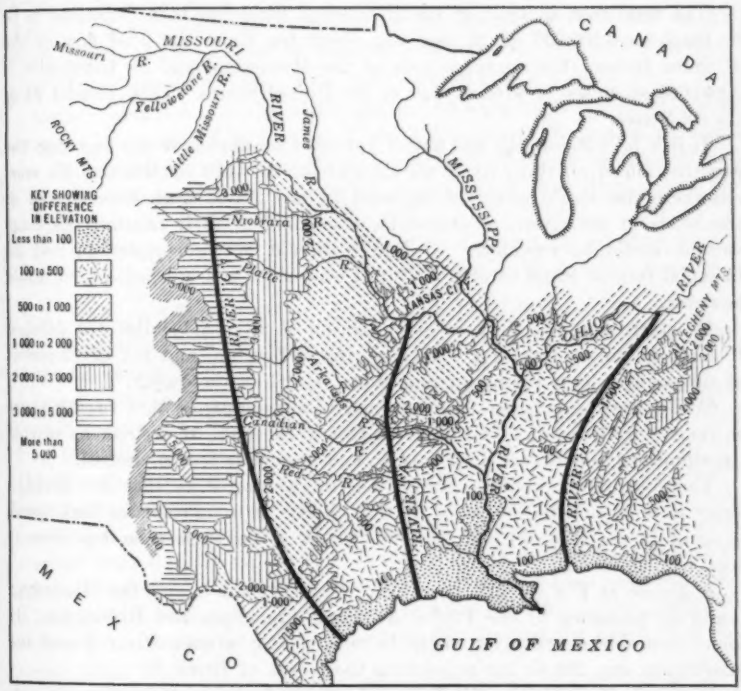

FIG. 56.—PLAN OF THE MISSISSIPPI RIVER VALLEY.

River B.—The work on River *B* should commence 100 miles, more or less, east of the mouth of the Mississippi, on the Gulf of Mexico; 100 miles from the mouth of River *B* its elevation would be 100 ft., tapping the Ohio River near Ashland, Ky., or near the Big Sandy River.

River C.—River *C* should have its mouth on the Gulf of Mexico about 500 miles west of the mouth of the Mississippi; 80 miles from its mouth its elevation would be 100 ft.; 120 miles, 500 ft.; 150 miles, 1 000 ft.; 450 miles, 2 000 ft.; 600 miles, 3 000 ft.; and 1 100 miles, 4 000 ft., where it would con-

nect with the Niobrara River. If found desirable River *C* could eventually be extended in a northwesterly direction to the Canadian border.

Engineers should control and utilize the forces of Nature instead of allowing such forces to create ruin and havoc as is usual in all cases where floods are not controlled.

To realize the potential values of such huge heads and volumes of water a comparison might be made with the Great Lakes: Lake Superior has a surface elevation of only 600 ft. above sea level (its bottom is 150 ft. below sea level), and the mean flow over Niagara Falls is about 220 000 cu. ft. per sec. When fully developed, the water power of the Niagara and St. Lawrence Rivers will be worth more than $2 000 000 per day.

The flood flow, in 1927, of the Mississippi River has been estimated to be as much as 3 500 000 cu. ft. per sec., about ten times the flood flow of the Niagara River. The drainage area of the Mississippi and its tributaries is 1 240 050 sq. miles, or 41% of that of the United States. This includes 24 of the 48 States.

It will be noticed that the writer has tried to emphasize the fact that the construction of all these rivers should start at the Gulf of Mexico. To start anywhere else would result in financial failure, for if work were started at the heads of the rivers, or elsewhere, instead of at their mouths, the excavations would be repeatedly filled up with silt before completion, and no financial returns could be obtained until each river was finished, which would be never.

By starting at the Gulf, as every section of 50 or 100 miles was finished it could be turned over for operation, for navigation, and the development of the country, thus giving an immediate return on the outlay.

While, for the purpose of estimating, it was suggested that twenty years be taken for River *A*, as a matter of fact work could, if desired, be started simultaneously on Rivers *A*, *B*, and *C*, and pushed as fast as desired.

The four rivers would naturally be interconnected so that low or high water in one could be controlled by the others. It is unusual for high water to occur in the Missouri and the Ohio Rivers at the same time, but when it does, trouble results.

A glance at Fig. 56 will show that Rivers *A* and *C* and the Mississippi would be connected by the Platte, Arkansas, Canadian, Red Rivers, etc. If found desirable a new river might be constructed between River *B* and the Mississippi, say, 200 or 300 miles from the mouth of River *B*.

Rivers *A*, *B*, and *C* should all be narrow and deep (say, 1 000 ft. wide and, at least 25 ft. deep) and should be regulated so that navigation could be carried on 365 days per year, instead of being periodically interrupted by both high and low water, as at present, to the tremendous financial loss of the country. All the rivers should have flat slopes and beds lined with concrete so as to avoid scouring of the banks, forming new channels, etc.

Silting basins should be constructed so that the millions of dollars worth of top-soil which is now lost (1 000 000 000 cu. yd. per year) can be collected and distributed where and when needed, irrigating and reclaiming waste land, etc. Two examples that follow illustrate the quantity of this silt:

First.—When building a bridge over the Ohio River near the junction of the three States of Ohio, Kentucky, and West Virginia, the latter two being separated by the Big Sandy River, the writer found low water at 6 ft. and high water at 106 ft., with no soil on top of bed-rock. A coffer-dam about 30 by 60 ft. was landed on rock, but was not sealed, before high water compelled stopping work for the winter. In the spring it was found that the coffer-dam, 25 ft. deep, was filled with silt and there was no means of telling whether it would have been filled more than once if it had been emptied as fast as filled.

Second.—In placing some pneumatic caissons for a bridge in the Missouri River, in South Dakota, the writer did not consider it feasible to dredge a channel to float the caissons from the shore to the pier sites, as the channels would fill up almost as fast as dredged; so platforms were constructed on piles from which the caissons were suspended while being built.

As the drainage area of the Mississippi and its branches is nearly 800 000 000 acres, the increased value of the land, due to navigation, irrigation, reforestation, and general development—to say nothing of the hydro-electrical possibilities—should so enhance the value of the land that these rivers would soon be fully paid for.

Furthermore, the mere undertaking of this project would tend to open up vast territories, to the benefit of the entire Continent of North America.

W. E. Elam,* M. Am. Soc. C. E. (by letter).—That section of the Mississippi River lying between Arkansas City, Ark., and Greenville, Miss., known as the Greenville Bends, is, in many respects, one of the most interesting on the river.† At this point a series of five great bends is lying so close together that 45 miles of the river are compressed into a rectangle of approximately 9 by 10 miles (see Fig. 57). Two points, 40 miles apart by river, are only 10 miles apart in a straight line. Local tradition tells of the Indian, who, having to travel from Greenville to Arkansas City, floated down stream 5 miles, cordelled (that is, pulled his boat) across Leland Neck; floated down stream 4 miles; cordelled across Tarpley Neck; floated down stream 4 miles; cordelled across Linwood Neck; floated down stream 3 miles; cordelled across Ashbrook Neck; and by this process was 30 miles up stream from his starting point without having to row up stream at all. Instead, he had floated down stream about 16 miles.

The Mississippi River Commission estimates the possible range at the upper end of the Greenville Bends to be 78.1 ft. Considerable revetment, as well as two long spurs built to prevent cut-offs, are found in this reach. Here, the greatest change in flow conditions has been caused by the construction of levees, and a large percentage of the discharge flows across necks during extreme high stages of the river. The upper section is one of the wide sections and the lower end, at Warfield, Miss., is one of the much discussed "bottlenecks" that does not entirely bear out popular ideas.

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* Asst. Engr., Mississippi Levee Board, Greenville, Miss.

† *Engineering News-Record*, June 28, 1928, p. 996.

A detailed study of the hydraulics of this reach would do much to clarify flood control problems. It is an ideal hydraulic laboratory and a few more data on bank-full and lower waters would furnish a complete study of the slope problem that must be solved to get and maintain an adequate channel. Enough information to determine the normal slope is essential, but even this will be of little value unless high and low-water channel lengths and locations are made coincident. Then, determination of the proper length of channel to lower the high-water levels will be possible. It must be recognized that this discussion is a study using "incidental data" and the results can only be accepted as such. The maps and other data used, are taken largely from published records of the Mississippi River Commission and are known to be very accurate; but some vital data are interpolated from known data that may not be close enough for final conclusions.

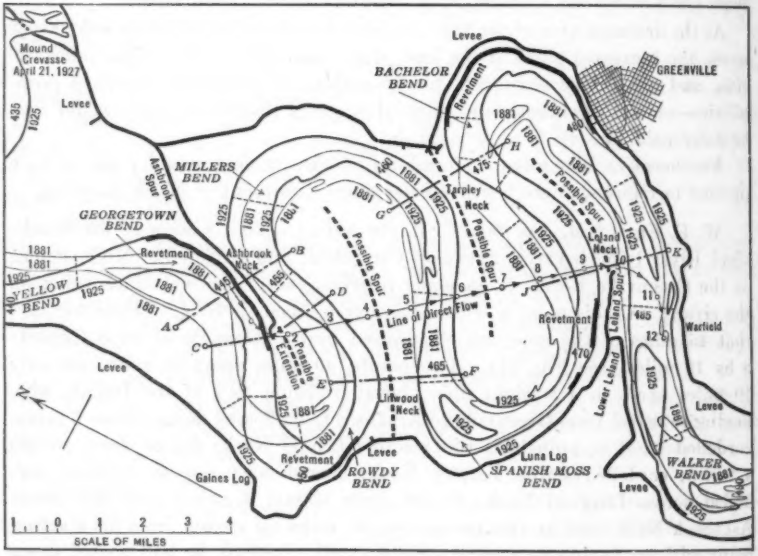

FIG. 57.—THE GREENVILLE BENDS, MISSISSIPPI RIVER.

Physical Features.—Under low and moderately high water conditions this part of the river is a simple channel, 56.74 miles long, from a point just below Arkansas City to Warfield Point about 5 miles below Greenville. When the water level gets above the natural surface of the ground at the neck sections, each neck becomes a weir, by-passing varying volumes of water to the bend below. The ends of these points are much higher than the neck sections and are constantly being built higher with deposit left by each extreme high water. On most, and probably all, of these points are small areas of land that build up almost as fast as the water rises. With a fall of less than 1 ft. from a maximum stage, land begins to appear on these points. The necks show some

tendency to build up, but owing to the depth and velocity of the water, the rate of deposit is not nearly so rapid.

The gauge relation between the Arkansas City and Greenville gauges has changed materially since 1881. Fig. 58 was constructed by plotting the differences in gauge readings for waters cresting between 10 and 30 ft. on the Arkansas City gauge. Two periods were selected, 1882 to 1887, inclusive, and 1922 to 1926, inclusive. The differences are plotted as plus quantities, using Arkansas City as a base. The Arkansas City gauge readings have probably been changing more than the Greenville readings, as there has been less change in channel length and flow conditions immediately below Greenville than between Arkansas City and Greenville. The curves could be drawn by eye through these points close enough for the purpose and show that for waters cresting in the low-water channel, and not flowing across points, the Arkansas City gauge averaged about 3¼ ft. higher between 1922 and 1926 than it did from 1882 to 1887. This is an average increase of about 1 in. per year for the period. Discharge observations are not readily available to compare the discharges over these gauge readings during these periods. Only two stations on the river, showing a maximum discharge in 1927, have been given by the Mississippi River Commission.* These are at Chester, Ill., and Carrolton (New Orleans), La., both far removed from this reach. This change of gauge relation is not in the right direction to indicate increased carrying capacity of the channel and must have some cause in the low-water channel because the writer has not considered any other water in determining the gauge relations. Two main causes that could produce this result are: (1) Lengthening of the channel; and (2) deterioration in the discharge capacity of the channel.

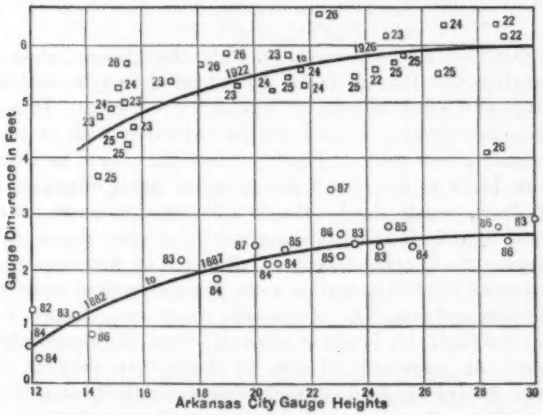

FIG. 58.—GAUGE DIFFERENCE BETWEEN ARKANSAS CITY AND GREENVILLE.

In 1881 and 1925, the length of the channel differs slightly from that given by the Commission. No very great refinement was attempted, but the distances were measured on the map as nearly along the line of the center of gravities of the sections as could be estimated by the eye. The length was

* Special Rept., Mississippi River Comm., November 28, 1927, Special Table No. 5.

estimated to be 48.34 miles in 1881 and 56.74 miles in 1925. The average slope from 1882 to 1887 was 0.218 ft. to the mile, and from 1922 to 1926, it was 0.247 ft. to the mile. This is a considerable increase of slope (in a longer channel) in view of the fact that two of these bends are practically completely revetted. This indicates that revetment alone cannot be depended on to flatten slopes and calls for an exhaustive study of the channel-way. An inspection of the profile, Fig. 59, shows that the channel is decidedly deeper in

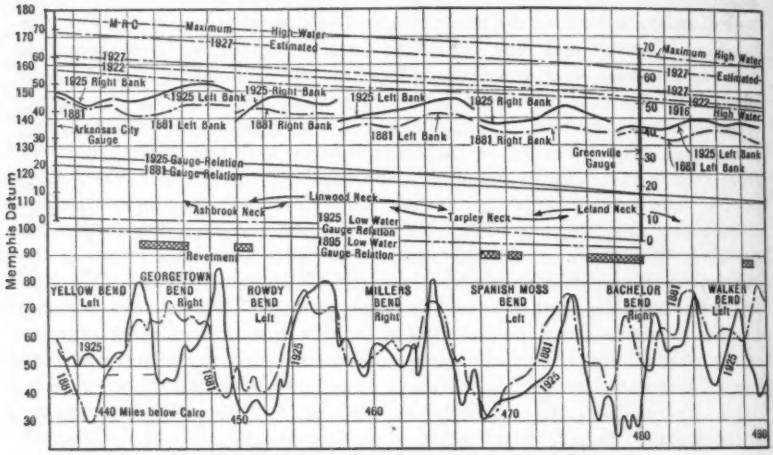

FIG. 59.—PROFILE ALONG CHANNEL, MISSISSIPPI RIVER.

those bends that have the most revetment; yet, the crossings show a tendency to be more shallow now than in 1881. This shoaling is more noticeable in the upper crossings and seems to have no relation to revetment. It is more probable that the crossings are shoaled during extremely high stages when the water is by-passing over necks. The lower crossings could be cleaned out by the draw from below as the river falls, while the upper crossings, beyond the effect of this draw, remain shoal. The area between the levees from Ashbrook Spur to Leland Spur is, in a sense, a reservoir in extreme stages with the main current following the channel except as influenced by flow across necks. The draw across Leland Neck is enough to make practically dead water in Bachelor Bend. The filling and emptying of this area create some changes from normal channel flow, although this is not so apparent from the gauge relation of the higher waters. An inspection of Fig. 60 shows that, in 1922, the average depth of water flowing across these necks was about 10 ft., and, in 1927, about 13 ft. The timber and undergrowth serve to check the velocity and maintain the slope at present. Whether this will continue at higher stages cannot be foretold. Shoaling of the channel in the bend also serves as a balancing influence and helps to maintain the slope.

The break in the slope of the low waters appears to be very much changed since 1881. At that time the upper part of this reach had a slightly steeper

grade, while now the lower section is the steeper and there is a decided hump in the slope through Miller's Bend which is behind Ashbrook Spur and is affected by the by-passing over Linwood and Tarpley Necks. This is shown on Fig. 59. The break in the slope in Bachelor Bend opposite Leland Point shows the presence of a constricted channel during high stages at this point, where it is 6 miles between levees, and also 9 miles above the "bottle-neck" at Warfield, where the levees are only 6 600 ft. apart. In extreme stages so much

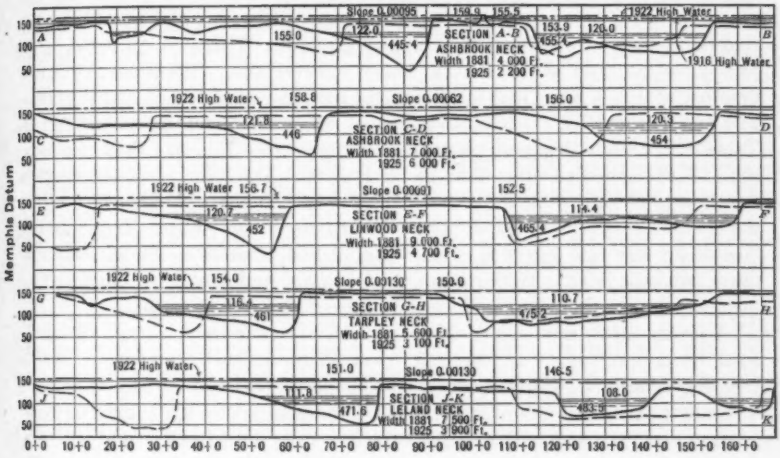

FIG. 60.—SECTION ACROSS NECKS, MISSISSIPPI RIVER.

water is passed over Leland Neck that there is practically no current in Bachelor Bend and the constricted channel is probably due to deposit dropped when the water is checked. The heavier materials would not be carried over the neck and would stay in the channel until dropped by the diminished current. It is also interesting to note that the channel is silting up at this "bottle-neck" and that the water lines show that there is no constriction at this point in either low or high water. When the water gets past Leland Neck it seems to have a free outlet. The only explanation for this is that the swift current that prevails here during high stages makes up for the lack of area, but this does not explain why the area is getting smaller. If it was known at what stages and under what conditions this deposit took place, some explanation could be given.

In order to determine the changes that have taken place in this reach, the Mississippi River Commission maps of 1881 and 1925 were reduced to a scale of 2 in. to the mile for working sheets. In the channel 10-ft. contours were drawn from soundings given on the maps. The zero contour on both maps is the estimated water surface of a crest of 20 ft. at Arkansas City passing to a crest of 17.7 ft. at Greenville, which was the gauge relation in 1881. It must be recognized that this discussion is a study of incidental data. The maps are known to be accurate, but this is a use not contemplated when they

were made; also, the exact determination of this water slope is impossible now. The data available are thought to be close and the line to be within a very few tenths of the actual slope.

Fig. 57 is a combination of the two working sheets, except that only 20-ft. contours are shown in the 1925 channel in order to give a better idea of the under-water channel conditions. The levees of 1881 can be ignored as they confined the water ineffectually and could have had little effect on channel conditions at that time. The bank lines of 1881 and 1925 are shown as the estimated water line at 20 ft. on the Arkansas City gauge and 17.7 ft. on the Greenville gauge, instead of the actual high bank. This is for a better comparison of that part of the channel under consideration. Revetment is shown practically as it is now (1928). The dotted lines show the divisions of the channels as separated for studying the changes which are given in more detail in Table 42. Lettered range lines across the necks (see Fig. 57) show the locations of the compiled cross-sections on Fig. 60. The "line of direct flow" is drawn as a straight line from the channel near the end of Ashbrook Spur to the channel just below Leland Spur. At extremely high stages water does not flow along this line where it crosses the channel; but it is the shortest distance from the inlet to the outlet of this reservoir and effects the channel flow in proportion to the volume of water by-passing over these necks.

A profile along this line is shown on Fig. 61. The lines marked "Possible Spur Extension" and "Possible Spurs" were drawn in such a way as to provide as nearly a uniform width of over-bank channel as possible and also to prevent excess by-passing of water over the necks. This would make the lengths of high-water travel very nearly the same as the low-water travel and would probably cause the channel to enlarge and perhaps scour out the shoaled crossings. To do this without reducing the length of the channel enough to compensate for the water that now by-passes over these necks, would probably raise the Arkansas City gauge to greater relative heights as compared with the Greenville gauge. Assuming that the 1881 channel was a natural channel, that length might be a very good guess. It is possible, though, that it might be less and perhaps cause more lowering of the Arkansas City gauge. This is a problem because, when the channel is enlarged, the slope may become flatter. With the by-passing and channel disturbances that now exist, it is not possible to state what the natural slope would be or even what it is now in this reach. From the profiles along the channel this appears to be a very flat section of the river when, in fact, it might be the reverse. There is also some doubt as to what the Arkansas City-Greenville gauge relation would be with a maximum or a 1927 estimated water confined between levees and greatly increasing the flow across these points and further disturbing the slope. The probability is that this break in the slope in Bachelor Bend would be accentuated and the fall across Leland Neck would be greatly increased. The Arkansas City gauge could hardly be reduced because it is just below the foot of a larger reservoir section very similar to this, except that the Arkansas and White Rivers empty into it.

Fig. 59 is based on the 1925 distances, in order to bring all possible points in common and compare changes. The depths of water, taken from sound-

ings, are the greatest in each line of soundings. At crossings, one sounding may be on one side of the river and the next sounding on the opposite side, with no regard to alignment. This method is thought to show more nearly the actual hydraulic conditions than that of following any alignment. Revetment is indicated as well as the approximate surface elevations along the neck sections immediately below each bend. High-water lines are shown as nearly as it is possible to estimate them from elevations along the levees on both sides of the river. The gauge relation in 1881 and 1925 is shown distributed. This distribution is based on only one water in each instance and then is adjusted to average gauge relations.

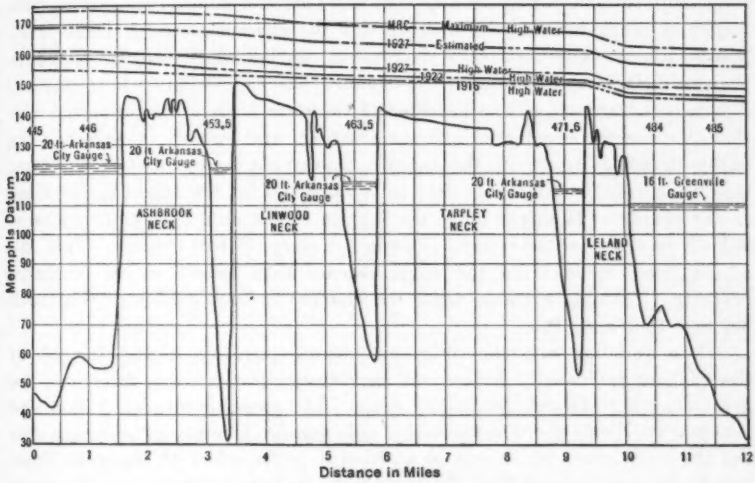

FIG. 61.—DIRECT-FLOW PROFILE.

The Ashbrook Spur was completed after the high water of 1916, and seems to have had some effect on the water lines, although not as much as the general public thinks. The 1916 water shows a depression at about Mile 450 (not shown in later waters) that is probably due to the local effect of the flow across Ashbrook Neck. The Arkansas City-Greenville gauge relation was 5.65 on that crest and 6.03 in 1922. This small increase could be attributed to the lengthened travel of the water in going around the spur instead of across the narrow neck, but lengthening of the channel is the more reasonable cause. During the 1922 high water, rapid currents were very noticeable near the outer end of Ashbrook Point both in the channel and across the point. This must have enlarged that channel to some extent. Scaling this slope on Fig. 59 shows it to have been about 0.17 ft. to the mile in 1916, 0.33 ft. in 1922, and 0.30 ft. in 1927. This is a closer agreement with anticipated results than was hoped for with the data used, and it confirms the assumption that the channel was first clogged up and then scoured out in 1922, restoring the normal slope nearer the critical point through an enlarged channel. This indi-

cates that 0.30 ft. to the mile is approximately the critical slope for this reach of the river. Using this as a basis, the 35 miles of river that existed in 1881 will give a difference of 10.5 ft. in the elevation of the water at these points, which corresponds exactly with the gauge relation curve. This was in a channel that was little affected by by-passing. The distance now is 43.5 miles and calculating the theoretical drop in the same way gives a difference of 13.05 ft. against an actual difference in 1922 of 13.86 ft. This indicates that flow across points helps little and may be a detriment. It also gives the hydrologist something to think about when he is hoping to get relief from high floods by widening the space between the levees. There is no section of the river more suitable for by-passing than this part and none where it is possible to get as much lowering by area outside the channel proper. There is no section where such short travel over necks can be substituted for such long channel distances. Shoaling of the channel must be attributed to by-passing. Navigation interests will be benefited if the river is improved in such a way that the channel is enlarged and the crossings are lowered. Compelling the entire discharge to follow the channel appears to be the first step. After this, some wide crossings may have to be narrowed for low stages by low dikes in the channel. If this is done, the 0.30 ft. to the mile may be found to be slightly in error for the critical slope in the more efficient channel. This will adjust itself by lowering the upper reaches with benefit to the discharge capacity. Table 42 gives in detail the changes in each section of this reach between 1881 and 1925.

Those familiar with the river can advance plausible reasons for any of the changes indicated. A few will be attempted where the differences are great enough to be definite in any sense. The crossing between Yellow and Georgetown Bends shows considerable enlargement (mainly by widening near the low-water line) which can be attributed to the effect of Ashbrook Spur. The Georgetown-Rowdy Bend Crossing seems to have lost area, and this might be due to material that came from above and to the fact that the force of the water was not enough to clean it out. Probably Ashbrook Spur is too short to create the current necessary to do this. Georgetown and Bachelor Bends, both almost completely revetted, show deepening and loss of total area but a gain in under-water area. This shows the good effect of revetment and the evil effect of allowing water to by-pass the channel when the current is best able to carry this material away. The Miller-Spanish Moss Crossing shows a loss of area and depth and an increase of width, probably due to island growth, aided by by-passing. In Table 42 the columns showing channel width are significant. With two exceptions (Rowdy and Miller's) the bends show a decided narrowing and deepening. The loss in area is mainly between low water and 20 ft. on the Arkansas City gauge, while on the average below the low-water line the area shows a gain. This indicates that revetment is mainly effective during low water. It is a little too much to expect to get this effect in high water when the force of the water is expended over the necks.

The writer defines critical velocity as that at which the material forming the bed and banks of the river is on the point of being moved by the force

TABLE 42.—CHANNEL DATA AND VARIATIONS BETWEEN 1881 AND 1925, ARKANSAS CITY, ARK., TO GREENVILLE, MISS.

Reach or section of river.	Length, in Miles.		At 20 Feet on Arkansas City Gauge.						At 0 Feet on Arkansas City Gauge.						Remarks.
			Average cross-section, in square feet.		Average width, in feet.		Minimum width, in feet.		Average cross-section, in square feet.		Average width, in feet.		Minimum width, in feet.		
	1881	1925	1881	1925	1881	1925	1881	1925	1881	1925	1881	1925	1881	1925	
Yellow Bend	1.70	1.83	113 500	107 500	2 410	2 360	2 100	2 200	69 000	65 200	2 140	1 985	2 000	1 950	Little revetment; shoaled.
Crossing	2.00	2.15	103 000	110 000	3 300	3 330	2 800	2 500	47 300	48 300	2 280	2 780	2 000	2 500	Depth decreased.
Georgetown Bend	3.95	4.48	106 200	101 000	3 760	2 960	3 000	2 300	41 800	50 300	2 550	1 840	1 800	1 250	4 miles of revetment; deepened.
Crossing	1.90	2.00	132 700	115 300	3 340	4 090	2 750	3 400	74 800	42 000	2 315	3 710	1 900	1 700	Depth decreased.
Rowdy Bend	4.15	6.17	116 200	118 600	2 570	2 600	1 800	1 900	65 500	73 600	1 915	2 060	1 700	1 700	1 mile + revetment; deepened.
Crossing	2.00	2.10	101 500	112 300	3 780	4 230	3 200	3 500	34 800	38 900	2 870	3 020	2 600	3 250	Depth decreased.
Miller's Bend	6.20	7.77	105 000	100 800	3 260	3 440	2 500	2 220	52 500	46 600	2 140	2 450	1 700	1 100	No revetment; little change in depth.
Crossing	2.00	2.00	119 500	116 500	3 600	3 980	3 000	3 500	51 800	45 000	2 700	2 940	1 800	2 300	Depth decreased.
Spanish Moss Bend	8.10	10.30	109 300	94 000	3 190	2 990	2 100	1 800	56 500	57 700	2 100	2 025	1 700	1 600	2 miles + revetment; deepened.
Crossing	2.00	2.00	135 000	93 000	3 670	4 050	3 000	2 700	37 000	43 400	2 700	3 020	2 300	2 300	No change.
Bachelor Bend	5.85	6.18	111 000	108 000	3 670	2 660	2 800	1 700	57 700	61 400	2 230	2 010	1 700	1 300	5-mile revetment; deepened.
Crossing U½	2.93	2.85	128 000	143 500	4 720	4 870	3 500	3 900	39 800	58 700	3 100	3 480	2 100	3 700	Above Leland Spur; deepened.
Crossing L½	3.16	2.94	131 000	112 000	4 670	3 920	4 400	1 900	45 900	56 800	3 100	2 180	2 100	1 600	Below Leland Spur; deepened.
Walker Bend	2.90	3.97	135 500	132 500	5 470	5 220	3 200	4 500	44 000	49 000	3 640	3 100	2 800	2 200	Much improved; deeper.
Totals or averages	48.34	56.74	113 200	109 000	3 620	3 420	.....	.....	52 200	54 700	2 580	2 430	.....	.....	.....

of the current. This velocity depends somewhat on the material and its resistance against disturbances. Critical slope is the slope or fall per mile necessary to create the critical velocity in the channel under discussion. The critical slope varies with every change of channel shape and channel material. The discharge channel, its size, and shape, depend on the critical slope. If the slope is less than the critical slope, the discharge channel will decrease in area and build up the upper reaches and the slope; if more, the channel will scour and restore the critical slope. The channel can scour either wider or deeper. If it widens, the critical slope becomes steeper; if it deepens the channel, the critical slope flattens. For this reason the river will have to be regulated to a very close margin in order ever to be able to tell exactly what results will follow these changes. While this is being determined the river can be regulated for all practical purposes and lowered gradually to obtain the required discharge channel. One problem to face when this is started will be to keep the sediment in motion, and this can probably be done by shunting most of it into old sections of the river that are to be abandoned, regulating this by a close determination of the slope. To get a channel for the Mississippi River large enough to carry a maximum flood, conditions will have to be created that will set up velocities in excess of the local critical velocities. Adjustment of length and alignment is the simplest method to get this condition. The entire flow must first be concentrated on the desired channel by properly located levees or spurs and further undesired bank caving prevented by revetment. The present concrete revetment used in the Third District, Mississippi River Commission, has about reached the stage where it can be depended on to hold almost any bank.

The Mississippi River has banks in the middle reaches that appear almost fluid at times. The hydraulic adjustments look almost hopeless when changing flow conditions create trouble at unexpected places; but the concentration of the flow along the same location for both low and high-water conditions will, no doubt, remedy this to a great extent.

The reach described is only one of many in the river from Cairo, Ill., to Baton Rouge, La. Beginning 10 miles above Arkansas City is a wide reach about 24 miles long, with an average width of 11 miles, that is even more complicated in its hydraulics. It has the Arkansas and White Rivers emptying into the upper end of it, and the back-water area of these two rivers adds complications to the problem. This area may be one of the places that cause the great range between high and low waters at both the White River and Arkansas City gauges. However, it is so disposed that, by constructing a spur about 17 miles southward from the Laconia Circle Levee, the back-water levels would be materially reduced and would give a more normal slope in the Mississippi River from White River, Ark., to Arkansas City.

All these considerations, then, support the following conclusions:

1.—Changes in gauge relations are due primarily to changes in channel length.

2.—Flow across points, or outside the channel proper, has little effect on slope, unless cut-off is imminent; then the slope in bend is flattened locally, thereby hastening channel deterioration and the cut-off.

3.—Flow across points tends to steepen the general channel slope and raises the upper gauges by checking the water in the bend and causing deposit which diminishes the channel efficiency.

4.—Channel elevations can be controlled to some extent by regulating the distance to tide-water.

5.—Channel capacity can be increased by making high and low-water channels coincident.

6.—Cut-offs require several high-water periods to develop and are seldom, if ever, accomplished suddenly.

7.—A study of channel changes throughout all the cycles of an actual cut-off is essential. Then, knowledge of facts will take the place of opinions.

R. W. DAVENPORT,* M. AM. SOC. C. E. (by letter).—The studies of Messrs. Woodward and Nagler show that apparently there has been no appreciable change in the flood run-off from the Des Moines and Iowa Rivers, caused by extensive development affecting one-third or more of the parts of the drainage areas considered. The authors conclude that agricultural drainage has had a negligible effect on the magnitude of either the total flow or the maximum discharge of the floods of the Mississippi River, since less than 5% of the water-shed has witnessed agricultural drainage operations.

Agricultural drainage undertakes to facilitate the escape of storm waters from agricultural lands and, consequently, when established on a large scale the possibility is suggested that its influence will be reflected by some modification in the regimen of stream flow from that existing in a state of Nature. The conclusion must be accepted, apparently, that particles of water falling on a drained area will pass through a cycle of behavior different from that which would have occurred before drainage. The determination by inductive processes of the net effect of the modified behavior on the resulting stream flow seems quite complex. Moreover, the characteristics of stream flow with respect to quantity and occurrence depend on numerous factors, and the effect of a change in a single factor, unless it is of predominating importance, may be obscured or practically obliterated by variations in the behavior of other factors.

If one can detect no appreciable effect of drainage works on the flood discharges of the Des Moines and Iowa Rivers, it may mean that the effect is relatively so small, in comparison with those discharges, that it is obscured by the variations of factors which are active in the production of floods. If one could determine what the net effect of drainage may be at lower stages where it can perhaps be more easily detected, it might throw some light on the nature of the effect at high stages.

A comparison of the characteristics of stream flow with respect to frequency of occurrence by use of the duration table or curve affords an excellent means of studying changes in the regimen of flow in a given stream by artificial causes. It occurred to the writer that such a comparison for the Des Moines and Iowa Rivers might be illuminating in this instance. Fig. 62 shows the results obtained for Des Moines River at Keosauqua and for Iowa River

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at Iowa City. The comparison is made in each case between the 3-year period, June, 1903, to May, 1906, and the 6-year period, October, 1917, to September, 1923. The investigation did not include the Des Moines River at Fort Dodge and Des Moines, because the records were not continuous through these periods. It is recognized that the periods studied are not long enough to show the frequency distribution with close accuracy, but it was thought that they might show evidence of any outstanding changes.

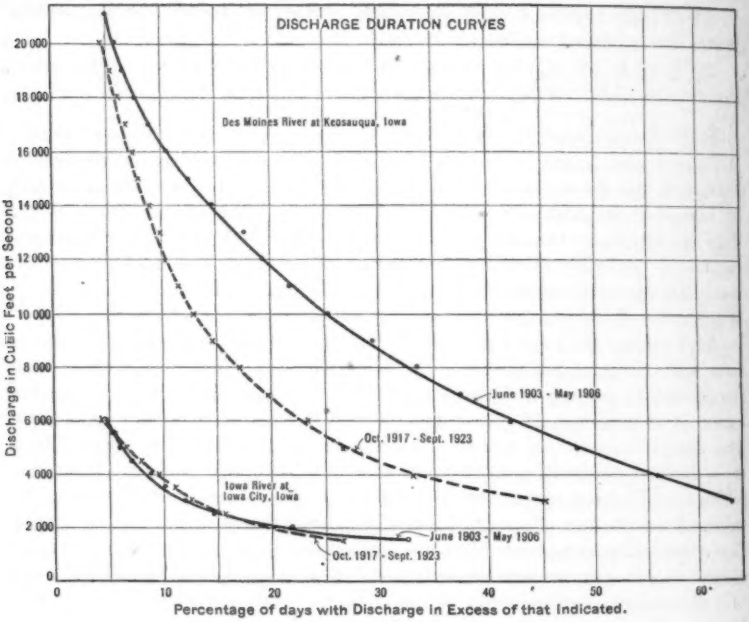

FIG. 62.

Because of inaccuracies involved in estimates of flow in times of ice cover, which, however, generally affect only the lower discharges, no attempt is made to define the duration curve below 3 000 cu. ft. per sec. for the Des Moines River, or below 1 500 cu. ft. per sec. for the Iowa River. Data are not shown for the highest discharges, 5% of the time being taken as defining the upper limit.

The curves for the Iowa River within the limits shown appear to be rather similar for the two periods. There seems to be a tendency for more frequent occurrence of lower discharges in the later period, 1 500 cu. ft. per sec. having been exceeded 33% of the time in the first period and 27% in the second. It may be that the records are not long enough to make this difference of special significance. If it is significant it is consistent with the deduction that the effect of drainage is to reduce ground-water storage, thereby taking away in part the reservoir available for maintaining low-water flow. It would,

of course, be a corollary to this that there must be a corresponding increase in the higher discharges. However, such increase need not have affected the highest discharges. In fact, it seems possible that in a flat country typical of those where drainage is practiced, a condition might exist comparable to that of a lake. If the ground and surface storage on the drainage area, like the storage on the surface of the lake, were utilized to high capacity, the escape of water from the storage on the drained area might be determined to a predominating extent by physical features largely or entirely unaffected by drainage works, in a similar way that the escape from the lake at any given stage is controlled by the characteristics of the outlet and is unaffected by artificial changes in the bed or shore of the lake at a distance from the outlet. Under the assumed condition, facilitating the escape of water from the drained area would become proportionally more effective as the stage of water fell and artificial drainage channels became relatively more influential in the discharge of water.

The results for Des Moines River show a difference in the characteristics of the occurrence of stream flow between the earlier and later periods, which is so great that it is not easy to reconcile it with the facts in an adequate way. The data indicate that the mean flow for the latter period is materially less than that for the former. It appears from superficial examination of precipitation records that there was apparently not sufficient variation between the two periods in regard to this factor to explain any material difference. This is supported by the fact that on the neighboring drainage area of the Iowa River there was no similar wide difference in quantity of run-off between the same two periods. Examination of the data on which the estimates of discharge were based, indicates no probability of any misinterpretation which could adequately account for so wide a variation. The writer's experience leads him to suspect that the apparent difference is probably greater than would be expected unless there had been a change in some important factor entering into the determination of run-off. However, he is unable to reconcile satisfactorily the extent and character of the apparent change in regimen with any probable effect of agricultural drainage.

Although failure to explain this difference may seem to make its consideration of little value to this discussion, it is presented as possibly having some bearing on the validity of conclusions based on the comparison of these records and with the hope that the authors will state from the standpoint of their intimate knowledge what they believe is the correct explanation of the matter.

E. W. Lane,* M. Am. Soc. C. E. (by letter).—Psychological conditions are often major items in flood control. Rarely, if ever, is a locality given adequate protection against floods until after a catastrophe has occurred with damages greatly exceeding the cost of suitable flood-control works. The City of New Orleans, is an extreme case of this condition. In spite of the frequently repeated example of the need of adequate protection in years past, culminating in the damage of several million dollars resulting from the artificial crevasse at Caernarvan during the 1927 flood, the danger from flood is

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as great as, or greater, in New Orleans than in any city of which the writer has knowledge. Such an obvious condition has not escaped the attention of some of the local engineers, but a frank statement of the situation is frowned upon by those who fear that it may frighten away business, or increase the apprehension of the citizens.

Some of the protection work has been excellently done. Levees with 50-ft. crown, 1 on 4 river slopes, and 1 on 10 land side slopes, inspire confidence. Even if the free-board is slight, there is ample opportunity for emergency topping which will hold stages higher than the poorer levees up stream. However, a chain is no stronger than its weakest link, and in this same main river levee are sections no larger than the old Mississippi River Commission standard, which the 1927 flood demonstrated to be inadequate even for the protection of agricultural land.

The greatest danger to New Orleans in the recent flood was not these main river levees, however, but in its second line of defense. Up stream from the city extends one of the weakest stretches of levee along the entire river. A breach there might flank the main river levees and flood a large part of the city. To guard against this contingency, a protection levee was built from the main river line to Lake Pontchartrain, connecting with the lake levees there. This is little more than a camouflage, however, for with a section of about 4-ft. crown, and 1 on 2 side slopes, with many street and railway openings to be closed with temporary bulkheads of piling or sand-bags, it is utterly inadequate protection for a city of 400 000 population against a flood of a month or more in duration. A large part, perhaps the major part, of the necessity for the Caernarvon crevasse arose from the weakness of this levee, which no one dared trust.

The construction of the Bonnet Carre Spillway, which will no doubt be finished as one of the first steps of the Mississippi flood-control project, will greatly reduce the hazard at New Orleans from Mississippi River floods, but it has no beneficial effect on what is perhaps a greater danger; that is, hurricane floods from Lake Pontchartrain. This lake lies north of the city and has an area of 600 sq. miles. It is connected to the Gulf of Mexico by two large passes. With the rapid growth now in progress, the city will soon occupy all the land between the lake and the Mississippi River. Much of this area is less than 2 ft. above Gulf level, some of it being below mean tide. To protect the city against rises in the level of this lake a levee, much of which is not more than 5 ft. high, extends along the shore. Through this levee, leading into the heart of the city, are several canals, the banks of which are only 2 or 3 ft. high. There are no gates or other protection works where the canals pass through the levee.

The Gulf coast is subject, at not infrequent intervals, to high tides due to West Indian hurricanes, which sometimes raise the levels 10 ft., or more, above the ordinary levels. New Orleans has been especially fortunate thus far in having escaped disaster from these storms. In 1915, the center of such a storm passed nearly over the city, but just enough to the proper side so that the wind blew from the city toward the lake, pushing back the water from the city. On the other side of the storm center, where the wind blew from the lake toward the

land, great damage occurred. On the main double-track line of the Illinois Central Railroad the rails were lifted bodily from a fill as high as the levee protecting New Orleans, and were thrown over into a swamp.

The situation at New Orleans is quite similar to, but even worse than, that at the towns along Lake Okeechobee, which were so badly damaged by the storm that passed over Florida in October, 1927, and which suffered so terribly in the storm of September, 1928. It is greatly to be hoped that the engineers who realize the situation will be able to secure support for adequate protection before "the horse is stolen".

C. McD. Townsend,* M. Am. Soc. C. E. (by letter).—As stated in the opening address of the Symposium, its object was to determine "in the further control of the river, to what extent shall levees, dredging, reservoirs, spillways, floodways, and other safety measures be used." The papers presented furnish many data on the subject, and elicit numerous differences of opinion.

Prior to the flood of 1927, the Mississippi River Commission attempted to solve a fairly simple problem: What was the best means of controlling the greatest flood that had then occurred in the Mississippi Valley? The Commission had a definite standard, the flood of 1882, which exceeded all those that had been recorded during a period of 142 years, and it arrived at the conclusion that a levee line of a computed grade was the cheapest method of regulating such a flood.

The flood of 1927, far exceeding that of 1882, has established a new standard; and this Symposium has not been devoted to a solution of the problem thus created, but to a determination of the best methods of controlling any flood which may occur. There is no precedent which can be used in a prediction of the greatest possible flood. Its determination depends on the mental peculiarities of the one who makes the prediction. Mathematics affords an excellent method of deducing conclusions from known premises, but when the premises are dependent on the vividness of the imagination, a mathematical analysis merely obscures the issue.

It has been clearly demonstrated that with the excessive rainfall which occurred in the lower valley during the flood of 1927, the average discharge curves determined from preceding floods, gave too great results, and that the factor of safety of 3 ft. adopted in determining levee heights was too small. It would be practicable to adjust those factors to the conditions which existed in 1927, but when it comes to substituting for the observed maximum discharge of 1 800 000 sec-ft. at Cairo, Ill., during the flood of 1927, such figures as either 2 250 000 sec-ft. or 3 000 000 sec-ft., the assumed mathematical solution becomes merely a flight of the imagination concealed under a mass of figures and mathematical symbols. The data furnished by this Symposium indicate that no standard can be established for determining what will be the characteristics of the greatest flood which can occur. Under such conditions the most logical proposition which has been advanced is to return to a grade line similar to that originally proposed by the Commission of 3 ft. above the heights

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attained by the flood of 1927, with spillways to relieve local conditions below the mouth of Old River; and to attempt by suitable means to guide the hand of Providence whenever extreme floods occur, so that as little damage as possible will be inflicted.

H. C. FRANKENFIELD,* ESQ. (by letter).—Acknowledgment is made to Dr. Brooks for his interpretation of rainfall data. His discussion of the causes of the spring rains of 1927 is excellent, and constitutes a very valuable contribution to the meteorological history of the great flood of that year.

Attention is also invited to the remarks by Mr. Eiffert relative to the effects of forests as factors in the prevention of floods in the great rivers, and more especially to the influence of the water content of the soil or its absorptive capacity. The controlling influence of the latter cannot be doubted, and knowledge of its character has always constituted a fundamental element of river-stage forecasting.

Relative to the need of co-operation with the U. S. Weather Bureau in the matter of rainfall records it has been stated in another connection† that "at different stations in the same region, records are made at different times of the day". This statement is only partly correct. The river-stage and rainfall records that are directly used by the River and Flood Service of the Weather Bureau are made at approximately the same hour throughout the country, with the exception of the districts west of the Rocky Mountains, where they are about two hours later, although uniform as to hour. The general statement applies to about 200 regular and about 900 special stations, a rather small number it must be admitted, but the largest possible under existing conditions. Other special services with about 350 stations in all also make rainfall measurements at approximately the same morning hour, making a total of about 1 450 stations. In addition, at more than 4 000 other special stations, known as Climatological Stations, where observers receive no compensation whatever, rainfall measurements are made each day about sunset.

Regarding the criticism that the requirement that the times of beginning and ending of precipitation should be recorded is largely ignored, it may be stated that these times of beginning and ending are accurately and fully recorded at the principal Weather Bureau stations, and special observers are instructed also to do this. However, the task is a difficult one for them to fulfill. They do not consciously and deliberately ignore their duties, but perform the service expected of them to the best of their ability. These are conditions which the Weather Bureau strives to overcome and improve as completely as possible, but, nevertheless, they are incidental to the co-operative nature of the personnel, who in reality render a highly valuable service at small expense. The observers, except at the regular stations, are local citizens who are paid only a few dollars per month, and they can neither be driven nor reprimanded for their shortcomings. Attempts in these directions are followed in many

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† Discussion of the Progress Report of the Special Committee on Flood Protection Data, by Arthur E. Morgan, M. Am. Soc. C. E., *Proceedings*, Am. Soc. C. E., May, 1928, Papers and Discussions, p. 1663.

instances by prompt resignations and inability to obtain successors. Moreover, in many isolated places, the mental attainments of the observers are not equal to the correct keeping of records, although almost invariably they are able to read a river gauge and measure rainfall correctly.

As to the criticism that "the Weather Bureau records are not more in accordance with engineering needs", the only answer that can be made is that the Weather Bureau has yet to receive from the Engineering Profession any intimation as to what its needs may be. It is scarcely necessary to add that any suggestions from such sources will be given most careful and sympathetic consideration, and will be adopted within the physical capacity of the Bureau.

One further thought is that most hydro-electric plants are provided with river and rain gauges as necessary adjuncts to the proper conduct of their business. Why cannot other lines of activity involving hydrological engineering in some form or other do likewise and furnish copies of their records on their own forms to the Weather Bureau for preservation and future use by those interested? The individual cost will be small and the resulting benefits mutual. Again, when a bridge is in course of construction, why cannot the builders install a river gauge with its zero set at the bed of the river, or at the bottom of flowing water, and its elevation determined by reference to a recognized bench-mark? This is sometimes done, and the gauges need not detract from the architectural effect in any way.

The problem of effective flood control is to be attacked from every possible angle, and its proper solution will demand the best constructive thought of the engineer. The preliminary work involves the gathering and assembling of a great many data relating to precipitation and river stages, many of which the Weather Bureau is now furnishing. The need for these data as a protective measure will continue during the period of active construction work, as the value of precise river forecasting will be so evident as to require no elaboration here.

In conclusion, the writer desires to emphasize the point that for the proper conduct of flood-control work as a whole more river-gauging stations and many more rainfall stations are absolutely needed, and every effort should be made to provide them.

E. F. McCarthy,* Esq. (by letter).—Mr. Eiffert has made a very fair analysis of some of the facts presented in the writer's paper. His summary as to the absorptive capacity of litter conforms closely with available records. However, the fact should be made clear again that this absorptive capacity of litter does not include the duff and humus which may be formed beneath the litter in some types of forest. Moreover, the forest floor owes its water storage capacity to the porous and loose condition of the forest soil induced by frost and biotic agencies under the protection of the forest litter. The water storage capacity of the litter itself is a minor part of the storage capacity of the forest floor.

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The place for reforestation is not on good agricultural land nor on good pasture land, and a ride through any of the hill sections of the Ohio Valley will convince the most skeptical that there are thousands of acres that are not now cultivated nor pastured where the cover of vegetation is too thin to protect the soil from washing. These acres are sources of silt which is filling the stream channels and reservoirs, creating engineering problems, while the hill land is being eroded into the waste-land class. Delay in restoring a timber cover will continue to reduce the possible returns from forest use of this land.

While a good sod cover is doubtless a suitable means for checking erosion, the fact must not be overlooked that such a cover can be maintained with difficulty on steep land and if once the soil is exposed, erosion may make the restoration of a sod cover an expensive undertaking.

WILLIAM KELLY,* M. AM. SOC. C. E. (by letter).—It was unfortunate that the studies of the Reservoir Board appointed by the Chief of Engineers, U. S. Army, and of which the writer was Chairman, could not have been completed before his paper was presented in October, 1927. However, the results of those studies were published† shortly after that date and were used by some of those discussing the paper. It appears proper, therefore, to consider them in this final discussion.

Considerable criticism has been launched against those studies, on the ground that they are inadequate and based upon incomplete data. They have also been attacked on the ground that the U. S. Corps of Engineers long ago took a stand against reservoirs and could not now be expected to reverse itself.

In regard to the first criticism, no claim for great accuracy has been made for the studies. They were no more than preliminary surveys, such as are made by most business concerns to determine whether projects offer sufficient promise to justify the expense of accurate surveys. It is claimed, however, that they were efficiently and honestly made in such a way that the probable errors would favor the reservoirs. Had they shown any reservoirs that offered even a remote chance of economical use as part of a solution of the Mississippi flood problem, accurate and complete surveys thereof would have been recommended. No claim is made that all possible reservoirs were considered, but the cheapest and most effective ones that were found, cost so much more for dependable results than the floodway plan recommended by the Chief of Engineers, that there is no reasonable hope that undiscovered reservoirs might overcome this difference in cost. There seems to be a growing element in this country that considers the United States Treasury inexhaustible and that believes the Federal Government is belittling itself or is far behind these scientific times when it proceeds along the lines that have been proved sound by years of experience not only in the Government service, but in business organizations. It is this element that is most liberal with volunteer advice. Engineering societies should be wary of permitting themselves to be led by the earnestness and eloquence of such advisers into advocating for the

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† Committee Doc. No. 2, Committee on Flood Control, 70th Congress, 1st Session.

Federal Government unwarranted expenditures, economically unsound projects, or the discard of proven organization. The writer is convinced that the preliminary surveys made are sufficient to justify the conclusion that the expense and delay necessary for more accurate surveys of reservoir possibilities for Mississippi floods are not warranted, and that conviction was finally shared by a Congressional Committee with a predisposition for reservoirs.

In regard to the second criticism, during thirty years of experience in the Corps of Engineers, the writer has never encountered a case where pride of Corps has affected the professional judgment of its members. On the other hand, such pride as the Corps has, is in its success in upbuilding its professional proficiency and integrity. The officers of the Corps are no different from other engineers. They are trained to seek facts and to form independent judgment therefrom and are never backward about expressing differences of opinion with each other. The fact that heretofore they have independently but uniformly reached a conclusion against reservoirs as a cure for Mississippi floods is a fairly good guaranty that under the conditions that have existed up to the present, reservoirs could not be economically used to relieve the Mississippi Valley.

Mr. Hall makes the statement that,

"To eliminate the damage caused by a flood it is only necessary to reduce or eliminate the crest wave, sufficiently; or, in other words, temporarily to detain in reservoirs that volume of water which would flow above the flood stage, as designated by the Weather Bureau."

Several others have made similar statements to the writer conveying the idea that, approached from this angle, the quantity of water involved is surprisingly small. It, therefore, seems pertinent to give some data on the size of the Mississippi flood waves. An inspection of the flood waves of 1922 and 1927 in the Mississippi Valley, when the flood was above the danger stage for about 45 days, should convince any reasonable person that the amount of storage to contain the water in the crest above danger stage of the possible maximum flood in the Mississippi River is very great. For illustration, the curves, Figs. 63, 64 (*a*), and 64 (*b*), might be designated "Superflood 1922 Type" at Cairo, Ill., Arkansas City, Ark., and Vicksburg, Miss., respectively. These curves are similar in shape to the 1922 hydrographs, but have the maximum discharge increased to 2 250 000 cu. ft. per sec. at Cairo, with the Arkansas and White River floods of 1927 added. The floodway plan of the Chief of Engineers will care for a flood of these dimensions.

With the present (1914) levee grades, the danger stages are: At Cairo, 58 ft.; at Arkansas City, 60.5 ft.; and at Vicksburg, 58 ft. The discharges of the river at danger stages vary considerably, but for a flood of the kind assumed they have been taken to be: Cairo, 1 810 000 cu. ft. per sec.; Arkansas City, 2 206 000 cu. ft. per sec.; and Vicksburg, 1 978 000 cu. ft. per sec.

A flood of the type shown on these curves would be above the danger stages, as follows:

At Cairo .....52 days with 32 000 000 acre-ft.
At Arkansas City.....51 days with 41 000 000 acre-ft.
At Vicksburg .....60 days with 59 000 000 acre-ft.

Nature provided the Mississippi River with a flood channel adequate for its needs. Man has found it profitable to take away from the river part of this channel. The question now to be solved is, "How much of this flood channel is it profitable to take?" Had it been possible to solve this problem when all the land belonged to the Federal Government it would have been much less complicated, because then benefits in one place would automatically offset damages in another place. Solution at that time was out of the ques-

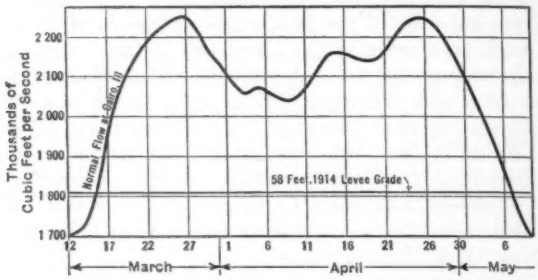

FIG. 63.—POSSIBLE MAXIMUM FLOOD (1922 TYPE) AT CAIRO, ILL.

tion and so the complications incident to ownership by different States, counties, levee districts, towns, and individuals now have to be met. Unprotected land in the valley is worth perhaps from $1 to $10 per acre. Protected land is worth from $75 to $150 per acre, and land in the towns is worth very much more. Each owner would welcome a flood plan that would promote his land to a higher value without expense to him.

The progressive confining of the river between levees has naturally been accompanied by a progressive rising of flood heights in the leveed channel. Heretofore, the funds available have never been sufficient to provide more

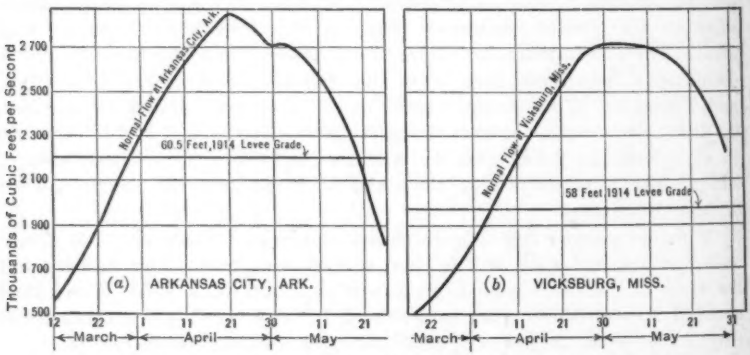

FIG. 64.—POSSIBLE MAXIMUM FLOOD (1922 TYPE) AT ARKANSAS CITY, ARK.

than the partial protection afforded by the present levee system. Property values and population have now reached a point where complete protection is demanded, and can be financed.

There seems to be general agreement that regardless of cost, the present levee heights on certain parts of the river should not be increased. On the other hand, additional channel capacity is essential unless the flood flows can be greatly reduced by reservoir storage. The studies made show conclusively that it is very much more economical to provide additional channel capacity by using a part of the natural flood channels formerly used by the river than to provide sufficient storage to make the present leveed channels adequate.

The development of such flood channels is a principal element in the plan recommended by the Chief of Engineers, and adopted by Congress. The lands within these floodways will be overflowed no more often than they have been in the past. Theoretically, therefore, their value should not be affected. On the other hand, if they could be protected from overflow, their value would be greatly enhanced. It is to be expected that the owners of land in the floodways will feel discriminated against because they are the only land owners in the valley not directly benefited; in fact, there are already indications that they will organize to protest. Whatever the equities in their case, nothing should be permitted to interfere with the development of adequate floodways, to supplement the leveed channels.

These floodways constitute the only safeguard for the remainder of the valley that can be developed within a reasonable time and, in the writer's opinion, they are the most essential part of the solution of the flood problem. If they are provided of adequate width, the flood problem can be solved promptly and economically; without them, no economical solution is possible.

It should be noted that this discussion is confined to flood control in the Mississippi Valley and does not apply to the very different problem of flood control on the tributaries of the Mississippi.

ARTHUR E. MORGAN,* M. AM. SOC. C. E. (by letter).—Probably the largest single engineering work now under construction in Europe is the reclamation of the Zuider Zee. A person examining the project is impressed with the ability and thoroughness with which every phase of this great problem is treated.

In former years there were marked differences of opinion as to proper methods, for the project is complicated, and is of the order of magnitude of the Panama Canal. To-day, however, there is substantial consensus of opinion as to the proper plans. This agreement has been reached, not through administrative discipline of divergent views, but by such a thorough working out of the problem that the essentially best solution is generally recognized and approved.

The Symposium on Mississippi flood control has brought clearly into view the fact that among representative engineers there is no general consensus as to the best solution of this great problem. The conflicts of opinion do not relate simply to details, nor are they confined to unrepresentative members of the Engineering Profession. They affect every significant phase of the problem, and they are held by engineers of every degree of standing and experience.

* Pres., Antioch Coll.; Pres., Dayton Morgan Eng. Co., Dayton, Ohio.

More than that, the orthodox and official program of protection, which was held and vigorously defended for half a century as being the only method to be considered, was suddenly abandoned as a result of the 1927 flood. Except for the spillway studies about New Orleans, La., this sudden change was caused by that flood, and did not otherwise follow any marked sudden increase of engineering understanding of the problem.

So fundamental was this change of plan that it required an absolute reversal of policy on main issues. For instance, the closing of the Tensas River Basin from the Mississippi was part of a deliberately matured policy of half a century, during which, according to the officials in charge of the Mississippi, that stream "was more thoroughly studied and understood than any other great river in the history of the world." The sudden change from this long-continued policy requires the undoing of the recently completed closure near the mouth of the Arkansas River, and the use of the Tensas Basin as the chief relief channel of the Mississippi.

There is no consensus of engineering opinion as to the proper methods to be used in controlling the Mississippi River. The present unity of policy has come by administrative discipline and not by analysis and clarification of the problem and a meeting of minds.

Where such divergent views are found concerning a great problem to which engineers have given so much attention over so long a period, the lack of unity can be explained in only two ways—either the problem is so inherently difficult that few engineers are capable of comprehending it, or else there is a lack of the essential data on which intelligent judgment can be based.

The writer believes that this latter explanation is the true one. For nearly fifty years the doctrine of "levees only" was held by the officials in charge as the only true faith, and, while a large amount of data was collected, it was with that particular plan in view, and many of the data necessary to provide an adequate criticism of that and all other plans, were not developed.

The hurried study made after the 1927 flood could not by any possibility furnish an adequate basis for the best treatment of the problem. The study of reservoirs is an example of this inadequacy. In examining the entire Mississippi water-shed in the course of three or four months with a hastily improvised staff, it was necessary to provide some general and arbitrary standards for measuring the value of reservoir sites. Colonel Kelly has described briefly what these standards are.

In the study of any particular reservoir site the use of these arbitrary standards would result in a failure to find the true possibilities of the reservoir in question. To multiply an inaccurate analysis a hundred times and to take the mean result as representative, is not dependable.

Discussing this point, Professor Woodward states,

"What Colonel Kelly calls dependable operation of flood-control reservoirs, is storage at a uniform rate for a period of 90 days, entirely irrespective of flood conditions on any other tributaries, or any other existing condition. This is perhaps the least efficient system of operation that can be imagined and still be called a system of rational control. It seems probable that reservoirs close to the region to be protected, can be operated three times as efficiently."

There is probably no one in America better equipped than Professor Woodward, both in technical training and in practical experience, for the design of flood-control reservoirs, and the writer believes that this quotation is a responsible and qualified statement of the value of the studies referred to as the basis for determining a great National policy.

The erroneous basis for calculating use is only one of the limitations of this study. The best possible sites for reservoirs, the best dam sites, and the most efficient depth of storage, can be discovered and determined only by a degree of thorough examination wholly beyond the possibilities of the limited time and personnel provided for this study.

The writer believes that an adequate study of reservoir possibilities on the main tributaries of the Mississippi still is to be made, and that to introduce into the creed of Mississippi flood control the doctrine that reservoirs are not feasible, would be premature.

Specifically, he believes that there is a strong probability that 10 000 000 to 30 000 000 acre-ft. of storage is possible on the Ohio and its lower tributaries, and that a development of even a part of this might entirely remove the need for the floodway in Missouri which is included in the present Army plan, and, in connection with the Tensas Basin spillway, or abandonment of certain Louisiana levees, might remove the necessity for any raising of the levee grade over that established before 1927. Adjustment to navigation requirements of such reservoirs on the Ohio River itself would require novel design, but the entire Mississippi problem is unprecedented, and requires unprecedented methods.

The public has often been given explanations of the levee failures in 1927 substantially in the words of Mr. Maltby,

"It is interesting to note that only one break occurred in Government levees built to standard height and cross-section based on the flood of 1912-13. It does not seem that the engineers made any very great error in designing the levee system as proposed."

As every engineer knows who has studied the situation, the Government standards referred to were totally inadequate to control the 1927 flood, even had every levee been built fully to standard. Statements like that just quoted, which were made repeatedly to members of Congress and to the public, represent a questionable type of defense.

The writer would like also to refer to the statement by Mr. Ayres,

"The writer particularly has in mind Mr. Morgan's paper. The case he makes against the Commission seems just a little bit too plausible. For example, he quotes the President of the Commission as having stated in 1912 that $73 000 000 was ample to do all the remaining necessary work on the levee system. This is no doubt a correct statement of majority opinion, but the writer happens to recall that the individual estimate of a civilian member of the Commission—C. H. West, M. Am. Soc. C. E.—at that time, was $250 000 000.

"This circumstance would seem to blunt the point of Mr. Morgan's sweeping denunciation on one score at least and leads to the belief that perhaps other scores would be similarly relieved of harshness if all the pertinent details were known. The impression of blind 'ancestor worship' so artfully created

by Mr. Morgan as the outstanding trait of the Commission personnel seems subject to discount on its face. Furthermore, it is an anachronism to state that the Commission failed to apply 1927 methods to the problems of the preceding century."

The writer made an exhaustive examination of the writings of the U. S. Army engineers and of members of the Mississippi River Commission during the past fifty years, as they appear in the *Transactions* of the Society, *Engineering News, The Engineering Record,* and *Engineering News-Record,* and the estimates in many of the Annual Reports of the Chief Engineers during that period. The figures he quoted are the representative, public statements of these officials, reported publicly and officially from year to year. No higher estimate by these officials was found in this search or omitted from the writer's original paper. What members of the Commission may have said in correspondence or conversation, the writer, of course, does not know. The estimates he quoted are those which Congress, the public, and the Engineering Profession were asked to accept as reliable on the authority of the officials in charge of this work. Such a statement as that quoted leads one to wonder whether at the present time there is the same divergence between the public and official estimates used for securing appropriations, and the private opinions of those most familiar with the problem.

Is it not remarkable, as Mr. Ayres points out, that the majority of the Mississippi River Commission—composed of Army engineers—should be more than 300% in error in their estimates, when a minority civilian member of the Commission had the basis for an estimate more than three times as large, and comparatively adequate? As to Mr. Ayres' remark about applying 1927 methods to the problem of the preceding century, is not foresight the very essence of engineering? The Mississippi River Commission made the flood of 1912 its standard to be protected against, and then announced repeatedly and unqualifiedly in its official annual reports that the completion to that standard would furnish permanent and complete protection from maximum floods. In this case was not ordinary engineering foresight lacking on a vast scale?

Mr. Ayres further remarks,

"It would seem almost self-evident that the first step in the solution of any scientific problem is the collection of adequate and indisputable facts; yet this basic phase of the Mississippi question has been undeniably neglected to a large extent, under pressure of restrictive legislation and limited funds. Lack of conclusive information doubtless explains in large measure the divergence of opinion so regrettably apparent, but for this, any indictment of the Mississippi River Commission must be leveled also at the entire profession."

Appropriations for securing the necessary information have been repeatedly introduced into Congress, and have been defeated at the specific objection of the Chief of Engineers on the grounds that adequate information is available. In 1922, the President of the Mississippi River Commission stated* that, "There is no uncertainty as to the outlay that will be necessary to make the levees safe for complete flood control * * *." That is not an excep-

* *Transactions,* Am. Soc. C. E., Vol. LXXXV (1922), p. 1479.

tional statement, but is representative of the attitude of the Commission. The fight to secure a hydraulic laboratory is another case in point. In Europe, the hydraulic laboratory has become fully accepted as an essential method for the study of hydraulic problems of every size, yet the establishment of a National hydraulic laboratory has been vigorously contested and for years definitely prevented by the Army engineers on the ground that it would be useless as an aid in solving the Mississippi flood problem. Only after continued and constantly increasing pressure has been exerted by civilian engineers in favor of a hydraulic laboratory is one now being established.

The Army engineers and the Mississippi River Commission in practice have held the purse strings, have dictated the policies, and have spent large sums in certain types of inquiries. They have definitely declined the cooperation of the profession as a whole, as tendered through the Society. A keen appreciation of the need for fundamental data on the part of those in charge of the work must, of course, precede any prospect for appropriations. In view of repeated assurances of the Mississippi River Commission and of the Army engineers that the data at hand were sufficient for the solution of the problem, the lack of adequate study cannot be attributed to the "pressure of restricted legislation and limited funds," nor can it "be leveled at the entire profession."

The writer has seen no discussion in this Symposium of the difficulties involved in constructing and maintaining the Tensas-Black River floodway from the Arkansas River to the Gulf. This area is almost everywhere densely overgrown with trees, brush, vines, and cane, no less dense than the jungles of Panama. If the area is left uncleared, its capacity for flood flow will be disappointingly small unless a very wide area is flooded, with corresponding damages.

If this floodway is cleared, leveled, and drained, so that it can be mowed with machines, that work will cost $100 000 000 to $200 000 000, and perhaps more. If it is roughly cleared and not maintained, the cost of clearing might be cut in two, but a single growing season will reproduce a young jungle with flow conditions not much better than before. To clear it by hand each year, removing the year's growth of willows and other brush, would entail an annual cost of probably more than $10 000 000 for a strip 5 miles wide and 300 miles long.

To maintain the surface of this area by pasturing, after adequate drainage works had been constructed throughout its length, would require 1 000 000 or 2 000 000 cattle. Any treatment of this floodway will be expensive far beyond any present estimate. The writer believes that the present floodway plans will gradually develop realization of difficulties that will involve the National Government in a maze of damage claims and expense that is not now foreseen.

A careful economic survey, such as should have preceded the adoption of any far-flung plan, might have proved the economy of abandoning most of the levee system on the west side of the river in Louisiana above Natchez, Miss., protecting the relatively small area in prosperous cultivation near Vicksburg, Miss., and paying damages for all other cultivated land for which protection is denied. Such a course would relieve the need for any further present levee

construction from the Arkansas River to the Gulf, and would enormously simplify the whole problem.

This step, combined with a few large dams on the Ohio River, might complete the reclamation for the next half century. The writer would be the last to advise such a program, or any final program, not based on an adequate study.

Mississippi flood control is one of the greatest hydraulic engineering problems in the world. Never was a Government better able, financially, to master such a problem. Summary and arbitrary administrative methods may be necessary to meet present emergencies, to prevent raids on the Treasury for impracticable schemes in all parts of the great water-shed, and to maintain the professional prestige of the organization long in control of policies on the Mississippi.

When those emergencies are met, however, is it not due the territory affected, the American public, and the Engineering Profession, that a thoroughgoing, inclusive, and masterly study be made of the Mississippi problem, drawing on the entire engineering resources of the country, to insure that Mississippi flood control will not be muddled through by the sheer weight of political power and public money, but shall be planned and executed so as to stand permanently as an engineering masterpiece?

MARCEL GARSAUD,* M. AM. SOC. C. E. (by letter).—The writer was deeply gratified to find the views expressed in the Symposium largely confirmed in the engineering report made to Congress by the Chief of Engineers, U. S. Army, early in December, 1927.

No engineer who has a reasonable grasp of the Mississippi River problem will question the desirability of further intensive study, or of the necessity of collecting additional hydraulic and physical data in connection therewith. The writer is firmly convinced, however, that sufficient studies have been made, and that enough data are now available, in order to determine the basic or fundamental considerations of a plan for the control of floods on the Mississippi from Cairo, Ill., to the Gulf, without further delay. It is evident that more adequate levees, compensating diversions, and bank stabilization are the three measures which constitute the most economical and most efficient method of flood protection.

While the writer advocated a spillway below New Orleans, La., he cautioned against permitting velocities greater than those obtained through New Orleans Harbor after the Caernarvon break in the spring of 1927. Since that time, enough data have been obtained indicating serious and threatening scouring at various points, due to a velocity of nearly 9 ft. per sec. This disturbance of the banks is viewed with alarm, and the writer, therefore, has completely endorsed the alternative suggested by the Chief of Engineers, namely, the diversion of 1 500 000 sec-ft. through the Atchafalaya Basin and the construction of a spillway above New Orleans at Bonnet Carre.

This plan would prevent the river from rising above the 20-ft. stage on the Carrollton gauge at New Orleans, even for a super-flood of 3 000 000 sec-ft.

* Gen. Mgr., Board of Port Commrs., New Orleans, La.

reaching the latitude of Old River, and would eliminate the objectionable velocities along the river front at New Orleans.

C. E. GRUNSKY,* PAST-PRESIDENT, AM. SOC. C. E. (by letter).—There is one matter connected with a determination of the rate of discharge at the peak of a flood which does not appear to have received sufficient attention in the past. For example, a period of about ten days intervenes between the time a flood crests at Cairo, Ill., and the time of the arrival of the peak of the flood at New Orleans, La. If a sudden rise at Cairo is taken into account, it may be nearly, if not quite, ten days before the effect of this rise begins to be felt at New Orleans. In the meantime what has become of the water which, during these ten days, has been added to the original flow of the Mississippi at the upper station?

The answer is simple. The entire discharge of water in excess of the normal flow of the river, at the end of the ten days, must be in temporary storage, either in the river channel or in the basins which receive its overflow at points between Cairo and New Orleans. It is obvious that the river at New Orleans remains high longer under the influence of an up-river flood wave than at Cairo. This must be so, because the river will remain relatively high during the time which it takes the channel above New Orleans to discharge the water temporarily stored therein. It follows, from this consideration, that channel storage is an important factor to be taken into account when the effect of a flood from a tributary or from up-river parts of the water-shed on the rate of flow at some down-stream point is to be determined. The writer has discussed this effect before† and, therefore, that analysis need not be repeated, but its application to such situations as that of the Mississippi cannot fail to show the need of large facilities for channel storage in rivers which have a long course through broad valleys of comparatively slight fall. On the Mississippi River, for example, facilities for water storage must be provided below Cairo either between river levees, in overflow basins, or in so-called by-pass channels. In the Sacramento Valley, California, by-pass areas have replaced the natural overflow basins. In other words, the temporary storage between levees must be provided in a quantity that will cut down the river peak discharge to channel capacity, that is, capacity at highest permissible water levels between levees; or as an alternative, the discharge of the river in excess of such capacity must be allowed to escape.

Much has been said in this Symposium, and in the discussions thereon, relating to the effect of reforestation and the benefits that might be derived from partial flood control by storage in reservoirs. No conclusive demonstration of any dependable favorable effect of reforestation can be hoped for, and, in consequence, it should be assumed, for the present at least, that no benefit therefrom is in prospect. In many cases, reservoirs are very effective in equalizing stream flow, thereby cutting down the peak discharge and aiding in the prevention of disastrous high river stages. It frequently happens, however, as in the case of the Mississippi River, that great floods result from heavy rainfall on a comparatively small area of the river's water-shed. In such

* Cons. Engr. (C. E. Grunsky Co.), San Francisco, Calif.

† *Transactions*, Am. Soc. C. E., Vol. LXI (1908), p. 332.

cases, reservoirs that may be adequate to cut down to reasonable proportions the discharge from other parts of the water-shed, may be of but little or no help in the solution of the flood-control problem. Conditions such as those on the Colorado River, where large storage far down on the stream will make possible complete control of the flow from the upper river and its upper tributaries, are rare. However, even in the case of the Colorado River—despite the contemplated regulation of discharge by storage in Boulder Reservoir—the section below Yuma, Ariz., may go to extreme flood stages on rare occasions, as it did in 1905, by reason of heavy rainfall throughout the water-shed of the Gila River, which enters the Colorado River at Yuma.

The writer refers to these matters only to emphasize the fact so well known to the river hydraulician that no general laws on the subject can find universal application. Each river system requires special study.

Very little attention has been given in the discussion of the Symposium to the economical aspects of the problems involved. A consideration of these is usually quite as important as the solution of the engineering problems. Thus, for example, attention must be given, when planning works for the reduction of a river's peak stage, to the cost of acquiring rights of way, flowage rights, etc., as against that of absolute ownership of the affected areas. The writer is of the opinion that a mistake was made in California where rights were purchased to flood at will the so-called by-pass areas. All lands required for occasional submergence, such as the submersible lands in retarding basins and in by-passes, should be owned by the public. If the inundation of such lands is periodical or only occasional they may yet be available for planting to certain crops, and can be leased for such purposes. On the other hand, if privately owned, there will always be more or less uncertainty as to the attitude of the owner, particularly if the functioning of the flood-gates, movable sections of weirs, etc., is not absolutely automatic and unchangeable. Where human judgment must determine the period during which water should be allowed to escape from a river, for example, into a by-pass or into a retarding basin, the possibility of a land owner claiming damages by reason of error in judgment should be forestalled. This can be done only by eliminating private ownership.

SHERMAN M. WOODWARD* AND FLOYD A. NAGLER,† MEMBERS, AM. SOC. C. E. (by letter).—The various discussions of the writers' paper have contained numerous interesting and useful suggestions. The writers tried to deduce in an unprejudiced and impartial manner the most complete and reliable conclusions that could be drawn from the data that were available to them, but it is evident that the subject is not exhausted. It is to be hoped that the accumulation of future data will enable still more complete and convincing studies to be made.

Mr. Moyer's discussion of forecasting future frequencies of phenomena like rainfall and run-off shows the possibilities of mathematical analysis in this direction, but his empirical methods are too intricate to be followed readily by most readers, so that some further discussion of fundamental elementary principles seems warranted.

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† Prof. of Hydr. Eng., State Univ. of Iowa, Iowa City, Iowa.

Frequency or Probability of Recurrence of a Phenomenon.—Using annual rainfall records for the purpose of illustration, suppose records for 12 consecutive years are available. If these are arranged in order of size and are numbered—the smallest, 1, the next, 2, etc.—the last and largest will be 12.

No. 12 having occurred once in 12 years may be said, according to this record, to have a probability of recurring once in 12 years, or to have a frequency of 12 years.

Similarly, it might be said that No. 11 has been reached or exceeded twice in 12 years, and so has a frequency of 6 years. Likewise, No. 10 would have a frequency of 4 years; No. 9, a frequency of 3 years; and No. 7, a frequency of 2 years.

The general formula* for obtaining the frequency corresponding to any value is:

$$F = \frac{T}{T + 1 - N}; \text{ or, } N = \frac{(F - 1)\ T}{F} + 1 \quad \ldots\ldots\ldots\ldots\ldots (8)$$

Equation (8) seems fairly simple, but it deserves further consideration. While No. 7 has been reached or exceeded on the average once every 2 years, it could also be said that No. 6 has been exceeded six times in the 12 years, or on the average of once every 2 years. With this significance:

$$F = \frac{T}{T - N}; \text{ or, } N = \frac{(F - 1)\ T}{F} \quad \ldots\ldots\ldots\ldots\ldots\ldots (9)$$

For some uses the latter significance of frequency might be the one desired; in other cases, the former. Therefore, in the range, or band, or strip, between No. 6 and No. 7, there is a certain ambiguity or vagueness of meaning.

Suppose that the record becomes ten times as long. For each of the original single recorded values there are now ten recorded values. These will not be identical, but will be spread over the bands or strips between the original values in the 12-year record. Some of the new values will be smaller than the least of the original twelve records, and some will be larger than the greatest. If all the new values are arranged in order and it is desired to find the value of N corresponding to $F = 2$, for example, it will now lie in a choice between the sixtieth and sixty-first recorded value. The range or discrepancy now will be only one-tenth of what it was before. It is evident that as the length of record increases the discrepancy approaches zero. This suggests the question: Can a corresponding value for this final frequency be obtained from the original brief record?

If it be assumed that the original recorded values are so well distributed that each truly represents the middle value corresponding to its possible range, and probably this assumption is as reasonable as any, then it is easily seen that the formula for frequency becomes,

$$F = \frac{T}{T - N + \frac{1}{2}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (10)$$

Using, again, the original illustration of a 12-year record, Equation (10) gives, for the sixth record, $F = 1.85$, and for the seventh record, $F = 2.18$.

* For notation, see p. 910.

The fact that Equations (8), (9), and (10) are all different mathematically, is due in part to differences in definition of the meaning of the word, frequency, and in part to differences in assumptions as to the occurrence of natural phenomena. Each of these equations may be satisfactory for use in some circumstances. Which one should be chosen in any particular case is largely a matter of individual preference or judgment. The writers prefer Equation (10) for ordinary use. Mr. Moyer has evolved still another slightly different form of equation, (Equation (1)). All these different equations give practically the same result for F for small values of N. As N approaches T, however, the different values obtained for F may vary by a considerable amount. For example, in Table 39, the values for F, obtained from Equation (10), differ less than 1% from Mr. Moyer's values, except for the last two or three lines in each part of the table. For Line 8 of the first part of Table 39, Equation (10) gives a value of 16 for F instead of the value, 15; for Line 14 of the second part of the table, Equation (10) gives 28 instead of the 21 in the table. It is just the last values in the table which are known to be most uncertain of interpretation. It is evident that there is no limit to the variations in form that might be assumed for the empirical equation for F, and in the present state of engineering knowledge there is no method of demonstrating conclusively which is the best form.

The next important step in Mr. Moyer's method is the preparation of the graph paper for plotting values of Q and F so that the line obtained will approximate a straight line. The value of F is to vary from unity to infinity, and it is fairly evident that, if uniform scales for F and Q were used in Fig. 48, the resulting curve would be convex upward. In such circumstances it can always be hoped that by distorting the scale for one or both the variables it may be discovered that the curve can be transformed into an approximate straight line.

The scale for F on the upper edge of Fig. 48 evidently resembles the logarithmic scale as it appears on the ordinary slide-rule. A logarithmic scale, however, would have had the disadvantage that it would never reach infinity, although it is not apparent that this fact would be any great detriment in this case. Most investigators in such a situation would probably try plotting Q and F on logarithmic paper, or on semi-logarithmic paper, in the hope of finding a satisfactory empirical rule for distorting the F scale. Mr. Moyer adopted for his use the relationship represented by the rectangular hyperbola

With the hyperbola represented by the equation:

$$d = \frac{1}{F} \qquad (11)$$

$d = 1$ when $F = 1$, and $d = 0$ when $F =$ infinity.

By writing:

$$d = \frac{x}{x + F - 1} \qquad (12)$$

the curve has the same properties as in Equation (11) and, in addition, can be made to go through any other desired point by determining the appropriate

value of x. As may easily be found by trial, any constant value of x at about 25 will give a relationship between d and F rather close to that shown by the scales in Fig. 48, throughout much of their extent.

The equation,

$$d = \frac{1}{F^{\frac{1}{4}}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (13)$$

also gives nearly the same relationship between d and F as that obtained by Equations (5) and (6).

By adding two hyperbolic equations containing two constants, v and z, as Mr. Moyer does to obtain Equation (5), a curve is obtained which will go through any two desired points, in addition to the point at which d and F equal unity. It is to be regretted that he did not explain through what procedure he happened to fix upon the values of the constants, v and z, which he adopted to give Equation (6).

Mr. Moyer has displayed much ingenuity in deriving his empirical equations, but before these equations can be generally accepted it will probably be necessary to show two things: First, that these forms are better as regards simplicity, accuracy, and convenience, than any other equivalent or similar empirical equations that can be devised; and, second, that the results obtained by the equations are reliable and useful, as proved by some kind of comprehensive and conclusive test.

It is the writers' opinion that the periods selected for comparison are too short to average accidental variations in the frequency of flows of different magnitudes. Hence, while the conclusions drawn from frequency curves such as those prepared by Mr. Moyer, or duration curves such as those used by Mr. Davenport, tend to support the original findings with regard to the effect of drainage on floods, the opposite might have been a likely result from an alignment of the data covering a period of so few years.

In fact, the lines drawn on Mr. Moyer's diagrams do not appear to be determined so definitely by the few scattered points that other lines with different slopes and intercepts might not have been drawn fitting the data equally well. In one trial, the writers eliminated from the frequency data, all values of rainfall and run-off for the month of June, which originally had been selected by design, and not by chance, in using only those June months which had appreciable flood run-off. Better alignment of the curves was obtained in most cases, but different slopes and intercepts were secured, that indicated no positive effect of drainage. This is no criticism of the device used by Mr. Moyer or by Mr. Davenport, which might be applied in comparing two very much longer periods of record; but many factors which affect run-off may not have been averaged in the two periods or they may have been averaged alike in the two basins, as indicated by the duration curves prepared by Mr. Davenport.

In this connection certain comparisons of general hydrologic data are offered (Table 43) as being of significance in interpreting the results shown by the duration curves of stream flow prepared by Mr. Davenport.

The fact that the Iowa River water-shed received 2.27 in. more rainfall in the post-drainage period, while the Des Moines River water-shed received 1.51

in. less rainfall, might account for the difference in the comparative duration curves observed by Mr. Davenport. A comparison of the rainfall received during April to November, inclusive, excluding the winter period in the manner followed by Mr. Davenport, shows approximately the same difference in the relative quantities received by the basins in the two periods. In basins such as those of the Iowa and Des Moines Rivers, where the average annual run-off is only 20% of the precipitation, an additional inch of rainfall may have considerable effect on the run-off, especially if it is received in a concentrated unit in the spring or fall.

TABLE 43.

Item.	Average Values for Period:			
	June, 1903, to May, 1906.		October, 1917, to September, 1923.	
	Des Moines Basin above Keosauqua, Iowa.	Iowa Basin above Iowa City, Iowa.	Des Moines Basin above Keosauqua, Iowa.	Iowa Basin above Iowa City, Iowa.
Average annual precipitation, in inches	32.81	30.87	31.30	33.14
Average annual run-off, in inches	6.69	7.54	4.88	6.78
Average annual water losses, in inches	26.12	23.33	26.42	26.36

Average Temperatures in Iowa, in Degrees:

Month.	June, 1903, to May, 1906.	October, 1917, to September, 1923.
January	16.6	21.2
February	17.1	24.4
March	31.1	38.2
April	48.0	47.7
May	59.6	61.5
June	67.2	71.0
July	71.4	74.8
August	70.8	72.2
September	63.5	65.2
October	51.5	52.8
November	37.9	37.6
December	23.3	23.5
Year	46.6	49.2

The fact that average annual temperatures (and almost every monthly average temperature) were considerably higher in the post-drainage period probably accounts for the increased water losses in the latter period; but the periods are so short that other factors, such as storm magnitude, distribution, and location, might have quite obliterated any effect of drainage as shown by the duration curves. In the year, October, 1917, to September, 1918, the Iowa River Basin received 8.40 in. more rainfall than the Des Moines River watershed, and the Des Moines River Basin received 6.13 in. more rainfall than the Iowa River Basin in the year, October, 1903, to September, 1904. There were

more rainy days on the Iowa River Basin in the post-drainage period and fewer rainy days on the Des Moines River Basin. During the summer following the notable flood of June, 1903, the Lower Des Moines Basin received heavy rainfall which maintained high stages on the Des Moines River throughout the summer, while the undrained areas in the Upper Des Moines Basin and the Iowa River Basin received no abnormal amounts of rainfall and the Iowa River had relatively lower stages during the remainder of the year. It was the complexity of the effect of the many factors which influence run-off in general that led the writers to test the effect of drainage by the analysis of the individual floods.

The test applied to the rate of depletion was carried to flows as low as the average flow of the stream, which was not reached in some cases until twenty days after the flood crest. The water-sheds are so small that run-off from the flat areas involved in drainage would have been measured for several days at all the gauging stations. The Fort Dodge Station on the Des Moines River is so located within this area that run-off from drainage lands would certainly have been measured at or immediately following the crest of the flood. Hence, the writers found it difficult to agree with Mr. Moyer that drainage would have no appreciable effect on this index.

In regard to the question raised by Mr. Davenport as to the effect of drainage on the low-water stream flow, the writers feel that this would be fully as difficult to determine as the relations which they undertook to study. It is an interesting and important matter, but quite foreign to the subject of the paper. So many unsupported statements of opinion regarding the effect of drainage on stream flow have been made, with little basis of facts, by opponents and advocates of agricultural reclamation, that the writers hesitate to express mere opinions; but it may be worth while to point out that it is entirely conceivable that drainage may reduce low-water stream flow without any corresponding increase in high-water flow.

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Paper No. 1710

THE CONOWINGO HYDRO-ELECTRIC DEVELOPMENT ON THE SUSQUEHANNA RIVER*

By Alexander Wilson,† 3d, Esq.

With Discussion by Messrs. William C. L. Eglin, J. V. Hogan, A. W. Clark, H. A. Hageman, and Alexander Wilson, 3d.

For more than forty years the possibilities of power development in the lower course of the Susquehanna River have engaged the attention of engineers and capitalists. The first evidence of contemplated power development in this river is found in the Act of the Legislature of the State of Maryland which was passed in 1884, authorizing The Susquehanna Water Power and Paper Company of Hartford County to acquire, by condemnation, certain property necessary for the proper extension and development of its existing dam or any dam it might locate or build near that site.

This water-power development consisted of a small wing-dam in the bed of the river, an intake channel, head-gate, power plant, and tail-race. Part of this work is still in existence and comprises what might be termed the first water-power development of the Susquehanna River.

Until, however, the growth of the steam-generated electric systems in near-by cities had developed a market capable of absorbing a large part of the energy in the flow of this river, its power development was not economically feasible. In 1910, the Pennsylvania Water and Power Company completed a dam and power house at Holtwood, Pa., which has an installed capacity of 150 000 h.p.

Early in March, 1926, The Philadelphia Electric Company, through its subsidiary, Philadelphia Electric Power Company, and the latter's Maryland subsidiary, The Susquehanna Power Company, under Federal license approved February 20, 1926, started the construction of the Conowingo Dam and Power

* Presented at the meeting of the Power Division, Philadelphia, Pa., October 6, 1926.
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House, with an initial wheel capacity of 378 000 h.p. (594 000 h.p. ultimate capacity), situated about 2 miles below the highway bridge at Conowingo, Md. The energy generated in this plant will be transmitted to Philadelphia, Pa., at 220 000 volts to be utilized in The Philadelphia Electric Company's System.

As stated, the dam and power house are in Maryland, but the upper half of the reservoir and the greater part of the transmission lines will be in Pennsylvania. This required joint action by the Public Service Commissions of the two States. Also, as the War Department has ruled that the Susquehanna is a navigable river, a license from the Federal Power Commission was required.

Corporate Structure

As The Philadelphia Electric Company will use practically the entire output of the Conowingo development, it was necessary that it should control the operation of the plant. It was also essential, in order to make the securities of the project attractive to the investing public, that The Philadelphia Electric Company should guarantee its completion and be responsible for the payments on which the securities depend for support. As The Philadelphia Electric Company is not allowed by its charter to do business in Maryland, these requirements were met by arranging to have three subsidiary corporations, as follows:

The Susquehanna Power Company, incorporated in Maryland, will own all the physical property of the project in that State, comprising the dam, power house, and tail-race, and parts of the reservoir and transmission lines.

Philadelphia Electric Power Company, incorporated in Pennsylvania, will own all the physical property in Pennsylvania, this being principally lands for the reservoir, and also the greater part of the transmission lines. This Company also owns all the stock of The Susquehanna Power Company. All the voting stock of Philadelphia Electric Power Company is owned by The Philadelphia Electric Company, which also leases the transmission lines owned by Philadelphia Electric Power Company.

The Susquehanna Electric Company, which was formed for the purpose of leasing, for the term of the license, the properties of The Susquehanna Power Company in Maryland, under contract with The Philadelphia Electric Company, will operate the plant and will sell all energy generated to The Philadelphia Electric Company.

As set up, therefore, the Conowingo hydro-electric development is being made by and for The Philadelphia Electric Company, and when completed will be operated as a part of that Company's System. The construction of the development is in charge of the Engineering Department of The Philadelphia Electric Company.*

Construction Organization

The contract for the design and construction of the dam and power house has been awarded to Stone and Webster, Incorporated, which has sub-con-

* This paper was presented and published before the construction work was completed. The final status of the project is outlined in the closing discussion.

tracted the construction of the greater part of the dam to The Arundel Corporation of Baltimore, Md. This Corporation has also been awarded the contract for the relocation of the tracks of the Columbia and Port Deposit Branch of The Pennsylvania Railroad. The contracts for the design and construction of the transmission lines, and for the switching station on the roof of the power house has been awarded to Day and Zimmerman, Incorporated. Construction progress schedules have been carefully worked out, and it is expected that the initial installation will be completed so that power from Conowingo will be available to help carry the peak load of The Philadelphia Electric System in December, 1928.

Precipitation and Run-Off

With the exception of the St. Lawrence, the Susquehanna River Basin is the largest and most important on the Atlantic Coast, and embraces a total area of 27 400 sq. miles, which comprises 47% of the area of the State of Pennsylvania, 13% of the area of the State of New York, and 2% of the area of the State of Maryland.

The annual precipitation over this area, according to the records of the U. S. Weather Bureau, varies from 31.4 to 44.3 in., with a mean of 39.4 in. The run-off which eventually finds its way to the sea through the Susquehanna River, varies from 16.6 to 29.1 in. and averages 55% of the rainfall. The run-off is a minimum in August, September, and October, during which months it ranges from 5 to 30% of the rainfall and averages about 15 per cent. Like all other Pennsylvania streams, the Susquehanna River has a natural run-off which is extremely variable, both from day to day, or week to week, and from season to season.

High water frequently occurs in January from melted snow. Floods accompanied by ice gorges come usually in March and result in a high-water level, although with a lesser volume of flow than at times of clear-water floods caused by heavy rainfalls, occurring over the whole or a part of the watershed as late as June. In the late summer or fall, periods of low water are frequently noted.

During the past century there have been several great floods in this river, the most notable of which was that of June, 1889, which was coincident with, although not caused by, the Johnstown Flood, and which probably exceeded any flood that ever occurred in this stream. It is estimated that during this flood the flow reached a maximum of 730 000 cu. ft. per sec. Reliable official records on the flow of the Susquehanna River taken for the last 31 years at Harrisburg, Pa., show that the lowest minimum discharge there was 2 200 cu. ft. per sec. This occurred in 1909.

The head-waters of this river system are on the elevated plateau which separates the waters which flow south and east into the Atlantic streams from those flowing north and west into the Mississippi, St. Lawrence, and the Great Lakes.

From Northumberland, Pa., situated at the junction of its North and West Branches, the river has generally a fairly uniform grade of 2½ ft. per mile, except for local rapids between the Muncy Dam and Conewago Falls,

near York Haven. Below, or in the lower 40 miles of its course, the slope increases to an average of 5 ft. per mile to tide-water and the width of the river becomes contracted, narrowing into a gorge which, in places, is reduced to a width of 0.2 to 0.5 mile. In the last 27 miles the river drops from an elevation of 225 ft., with an average slope of 5.6 ft. per mile, causing a swift current which has worn a low-water channel of great depth in many places.

Along this lower section, the river has cut its way through a range of table-land, and its bed is walled by steep rocky bluffs on both sides, affording excellent foundation conditions for water-power developments. A part of the fall in this section, as noted previously, has been developed by the Pennsylvania Water and Power Company's Dam at Holtwood. The Conowingo Dam will develop the head from the Holtwood tail-race to within 4 miles of tide-water.

Maximum Elevation of Pool

By agreement with the Pennsylvania Water and Power Company, a pool elevation for the Conowingo Dam, 108.5 ft. above mean sea level, has been adopted. At this elevation the water will be backed up over a part of the tail-race of the Holtwood Plant, which has not yet been excavated, but which, when excavated, will result in the development of increased head and power at the Holtwood Plant. It was mutually agreed that this additional head could be developed more economically at Conowingo, and it was arranged therefore, subject to approval of commissions having jurisdiction, that the Conowingo Pool shall be maintained at Elevation 108.5, the Holtwood Company to share in the Conowingo Plant's gain accruing from the increased head.

Selection of Dam Site

With the pool elevation decided upon, it was desirable, in order to develop the maximum head, to locate the dam as close to tide-water as possible. Here, however, the relocation of the tracks of the Columbia and Port Deposit Branch of the Pennsylvania Railroad Company, on the east bank of the river, imposed a limitation, in that it was necessary to provide a satisfactory grade from the present roadbed at the north end of Port Deposit to the elevation of the relocated tracks above the Conowingo Dam. A grade of 0.35% was finally accepted by the Railroad Company.

Five possible sites for the Conowingo Dam were examined and an exhaustive study was made of the head and power available and the total cost of development at each. The site as finally adopted is approximately 2 miles below the Village of Conowingo, and is far enough north of Port Deposit, Md., to permit the use of the accepted maximum run-off grade on the railroad. A reservoir having an area of approximately 14 sq. miles will be formed. The hills on either side of the river at this location form natural abutments, that on the Cecil County, or east, side rising to an elevation of 250 ft. above sea level, and that on the Hartford County, or west, side rising to an elevation of 155 ft.

The river bed and the banks, to a height well above pond level, are of granitic formation. During the autumn of 1924 twenty-six core borings were

made along the line of the up-stream face of the dam between the two abutments. These were drilled to depths varying from 5 to 30 ft. below the rock ledge, the average depth being about 15 ft. All these cores showed firm hard granite or gabbro. In addition, four borings were made to a depth of 100 ft. below the rock ledge, and these also showed hard rock for their entire length. This rock, when crushed to suitable sizes, is excellent material for concrete. On the east bank there is ample space for the erection of the construction camp and plant and for the storage of materials. Transportation facilities are supplied by the Columbia and Port Deposit Railroad.

The main channel of the river at this site is along the west bank. The power house, therefore, is being built at this end of the dam. Space for the construction plant on this side of the river is somewhat limited, but an old canal which formerly operated on this bank has been partly filled to provide space. To supply transportation facilities for the power-house construction, it was necessary to build approximately 10 miles of railroad to connect with the Pennsylvania Railroad System at Havre de Grace, Md. The tow-path of the old Tidewater Canal afforded an excellent roadbed, requiring very little grading, and within three months of the start of construction work this railroad was in operation.

Dam and Power House

The west abutment of the dam is in a projecting hill of rock, into which the retaining wall section, extending 140 ft. from the power house to the abutment, will be built to form an adequate seal for the impounded waters. The headworks for the power house, providing for eleven main units and two station service units, then extend 950 ft. to the beginning of the spillway section, which is 2 385 ft. long. From the east end of the spillway the retaining wall section continues 1 200 ft. to the east abutment, which also serves as the abutment for the highway bridge over the relocated tracks of the Columbia and Port Deposit Railroad. The seal at this point will be made by a core-wall extending up stream into a heavy embankment at the side of the railroad. As a further protection to the railroad, a core-wall will also be extended from rock to the underside of the ties, between the abutment and the east bridge pier.

The dam is solid masonry construction of the gravity type, founded on rock, at an average elevation 15 ft. above sea level. The roadway of the highway bridge on the dam, will be at Elevation + 114, with parapet walls 5 ft. high on each side. The width at the bottom of the retaining wall section is 67 ft., and at the spillway section, 85 ft. The spillway section, designed to take care of floods up to 880 000 cu. ft. per sec., will have a fixed crest at Elevation + 86, on which will be mounted, for the purpose of regulating the level of the storage reservoir, fifty movable crest gates. These gates will be constructed of structural steel and will be equipped with two roller trains on each gate and with sealing provisions along the vertical edges and bottom. They will be 22½ ft. high by 40 ft. long, and will weigh about 42 tons each. There will also be three regulating gates, 10 by 40 ft., of similar construction, adjacent to the power house. The operation of these gates will be effected by means of three electrically operated, traveling gantry cranes, runway for which has been provided

on the top of the dam and power house. Each of these cranes has been designed for a capacity of 60 tons. Each crane will be equipped with two main hoists of 30 tons each, mounted on a single trolley and operated by the same motor. The hoisting speed will be 10 ft. per min. and the gantry travel speed, 300 ft. per min. A generator driven by a gasoline engine will be installed on one crane for emergency use.

To insure water-tightness of the foundation, grout holes are being provided on 10-ft. centers on the up-stream face. Contraction joints in the dam will be sealed with copper sheets bent Z-shaped, the top and bottom of the Z being built into the abutting sections of the dam.

At the bottom of both the retaining wall and spillway sections, a continuous drain, 10½ ft. from the up-stream face, is to be installed with transverse drains leading to the down-stream face every 45 ft. Piers for the support of the highway bridge and crest gates will be constructed on 45-ft. centers and will be 7 ft. wide.

The power house, as noted, is adjacent to the west shore of the river. The head-works provide intakes for eleven main units and two station service units. By locating the top of the intake openings 40 ft. below the pool level, protection from floating ice has been secured without the expense of constructing the usual rock-fill and skimmer arch to protect the forebay. Structural steel trash racks, in sections, 14½ by 24 ft., will be provided for all units, including the station service units. Sectional head-gates, 14½ by 25 ft., complete with bronze sealing strips, will be provided back of the trash racks for use in case it is desired to unwater the intake passages. The sectional head-gates and trash racks will be handled by the gantry traveler on the dam.

The deepest excavation for the power house will be to Elevation — 20, as required for the construction of the draft-tubes for the Allis-Chalmers water-wheels. The superstructure will be of concrete, with a structural steel frame. The high-tension switching station will be on the roof.

Power-House Equipment

Seven main water-wheel units will be installed, four to be supplied by the Allis-Chalmers Manufacturing Company and three by the William Cramp and Sons Ship and Engine Building Company. Each unit will be designed to have a capacity of 54 000 h.p. at full gate, at 89 ft. normal net head, and 81.8 rev. per min., and will have a capacity of 50 000 h.p. at the point of best efficiency. They will be of the vertical-shaft, single-runner, Francis, reaction type, with plate-steel scroll case and governors of the actuator type, and will be the largest units, in physical dimensions, built to date (1926). The runners will be of cast steel, cast in not more than four parts. Each unit will have a structural steel pit liner or generator support capable of carrying the weight of a 40 000-kv-a. generator.

The governors will have gear-driven flyballs and will be operated by oil pressure. There will be one oil-pressure system for each pair of units. An oil storage and purification system will be provided for all the governors and bearing oil for the seven main units and two station service units.

Two station service water-wheel units, supplied by the S. Morgan Smith Company, will be provided, of 1 900 h.p. each, operating at 89-ft. normal net head and 360 rev. per min. The runners will be of the single, Francis, reaction type, equipped with vertical shafts, plate-steel scroll cases, governors, oil-pressure systems, and appurtenances.

With each main water-wheel unit, a vertical-shaft, butterfly valve, 27 ft. in diameter, complete with operating equipment, oil-pressure system, and accessories, and capable of operating under an 84.5-ft. head on the center line of the valve, is to be installed in the water passages to the runners. Each valve will have a cast-steel housing and a wicket of cast steel, a forged steel shaft, and a plate-steel penstock liner extending 12 ft. up stream. These valves will be operated by oil pressure from a central oil-pressure system.

Two butterfly valves, 6 ft. in diameter, will be provided for the station service units. They will have horizontal, forged steel shafts, cast-steel wickets, cast-iron housings, and plate-steel penstocks connecting the butterfly valves to the scroll cases of the units. A motor-driven operating mechanism will be provided, capable of operating each valve under a normal and maximum head at the center of the valve of 88.5 ft. The butterfly valves will be furnished by the same manufacturers as the respective water-wheel units.

Seven main generators and two station service generators will be installed; four main generators to be provided by the General Electric Company and three main generators and the station service generators by the Westinghouse Electric and Manufacturing Company. Each main generator will have a capacity of 40 000 kv-a., generating at 13 800 volts, 3 phase, 60 cycle, and 81.8 rev. per min., will be direct-connected to a main water-wheel unit, and will be provided with a direct-connected, 715-kv-a., auxiliary, alternating-current generator and a small direct-connected exciter for the auxiliary generator. A separate motor generator set will supply the excitation current for the main generator.

The weight of the rotating elements, together with the hydraulic thrust, will be carried by thrust-bearings on top of the generators. In the generators provided by the General Electric Company a thrust-bearing of the spring type will be used, while the Westinghouse generators will have Kingsbury thrust-bearings. The guaranteed efficiency of these main generators will be in excess of 97.0% at rated capacity.

Each station service generator will have a capacity of 1 600 kv-a., generating at 460 volts, 3 phase, 60 cycles, and 360 rev. per min., will be direct-connected to one of the station service water-wheel units, and will be provided with a direct-connected, direct-current exciter and Kingsbury thrust-bearing.

Stationary power transformers will consist of four banks of three transformers, each having the following rating: 26 667-kv-a. rated capacity, 29 333-kv-a. overload rating, single-phase, water-cooled, 13 800 low-tension voltage, and 127 000/220 000 volts Y at the high-voltage terminals. The guaranteed efficiency at rated capacity is in excess of 99 per cent.

Two power station cranes will be installed in the generator room. They will be of the 4-motor, electric traveling type, each equipped with a 150-ton main hoist and a 25-ton auxiliary hoist. Both the main and the auxiliary

hoists will be installed on a single trolley. The hoisting speed will be 6 ft. per min. and 30 ft. per min., respectively. These hoists will be equipped with alternating-current motors, operating at 440 volts, 3 phase, and 60 cycles.

Two 50-ton transformer traveling cranes will be installed for separate or combined operation. They will be of the 3-motor type, having a hoisting speed of 10 ft. per min. The motors will use alternating-current power at 440 volts, 3 phase, and 60 cycles.

One 15-ton machine-shop traveling crane of the 3-motor type, equipped for control from the floor, will be installed. The electrical characteristics of the motors will be similar to those of the other cranes. A small gantry crane will be installed on a runway over the stop-log slots on the down-stream side of the station, to be used for handling the stop-logs.

The tail-race for the seven main units and the two station service units of the initial development, will require the excavation of approximately 400 000 cu. yd. of rock, of which it is expected a considerable part will be crushed and used as concrete aggregate.

Progress Schedule

The progress schedule contemplates the completion of the initial project in two low-water seasons. Construction camps have been established on each side of the river, that of the The Arundel Corporation, which is constructing the greater part of the dam, being on the east side, and that of Stone and Webster on the west side.

During the low-water season of 1926, a coffer-dam, enclosing the power house and west branch of the tail-race, was constructed, to a height such that it will not be topped by any flood of less volume than 400 000 cu. ft. per sec., so that work on the heavy excavation within the enclosed area can be continued through the spring floods of 1927; also, a coffer-dam has been constructed from the east bank of the river out to within approximately 700 ft. of the power-house coffer-dam. The schedule requires that the dam foundations within the area enclosed by this coffer-dam be poured up to Elevation 22 with alternate sections of spillway up to Elevation 30, or higher, by January 1, 1927. The latter coffer-dam has been built with stop-logs alternating with the rock-filled cribs, so that as soon as the alternate sections are poured to Elevation 40, the stop-logs can be removed, to allow additional passage for the spring floods.

During the low-water season of 1927, the coffer-dam from the east bank will be continued over to the power house, the river flow being taken care of through the stop-log openings in the 1926 coffer-dam and the alternate sections of spillway at Elevation 22. As soon as the foundation of the dam can be poured in this coffer-dam, with alternate sections to Elevation 40, the stop-logs in the coffer will likewise be removed.

As the piers for the highway bridge will be included in the alternate sections of the spillway, which are to be carried up ahead of the closure sections, these must be pushed to completion so that the highway bridge on the dam can be opened to traffic, and thus permit the removal of the existing highway bridge at Conowingo.

By the close of the low-water season of 1927, the head-works of the power house should be completed. The relocation of the tracks of the Columbia and Port Deposit Railroad must also be completed and in operation by this date. In the fall of 1927, closure of the spillway sections will be commenced and is scheduled to be completed before the spring floods of 1928. During final closure of the spillway, it is planned to pass the river flow through light 10 by 18-ft. sluices built through the dam. These will be closed finally by means of specially designed gates.

Construction work was started on March 8, 1926, and excellent progress has been made on the work on both sides of the river. The work of relocating the tracks of the Columbia and Port Deposit Branch, was commenced on May 25, 1926, and is also making very satisfactory progress.

The construction of the dam and power house will require approximately 650 000 cu. yd. of concrete, of which, 400 000 cu. yd. will go into the dam. For the construction of the dam, a steel construction bridge with a deck at Elevation 60, is being built on the toe of the apron. On this bridge are three self-propelled, electrically-operated, gantry travelers, each equipped with a 15-ton derrick and a concrete hoist tower. There are also three standard-gauge railroad tracks on the bridge, and the gantry cranes are constructed so that cars on all three tracks can pass under them. The concrete-mixing plant on this side of the river is designed so that the concrete is discharged from the mixers directly into cars at the bridge elevation. Gasoline locomotives are being used to transport the concrete. Practically no rock excavation for the dam foundation has been required, it having been found necessary only to clean off the over-burden, remove the loose rock, and clean out the joints in the ledge rock.

For the construction of the power house and tail-race, the excavation of approximately 360 000 cu. yd. of rock is necessary, part of which will be crushed and used as concrete aggregate. A suitable crushing plant has been installed, with facilities for storing the crushed rock. In the placing of concrete for the power house, it is planned to use six steel towers, 250 ft. high, with 16-in. chutes.

A total force of approximately 3 000 men is now engaged on this work. It is expected that the first unit will be ready for operation about the middle of 1928, and that several additional units will be ready to help carry the peak load in Philadelphia that year.

DISCUSSION

WILLIAM C. L. EGLIN,* ESQ.—Any one who has never been intimately connected with a project of this nature, cannot appreciate the amount of preliminary work that must be done before actual construction can be commenced. If the preparatory work is not done carefully and thoroughly, disaster may overtake the project before it is completed.

In this project there was an enormous amount of preliminary engineering investigation and study required. Many problems of legal procedure and authorization, involving the Federal Power Commission, the Public Service Commissions of both Maryland and Pennsylvania, State Highway Commissions, County Commissions, Water Boards, Sanitary Boards, and numerous others, had to be satisfactorily disposed of before the project could be launched.

Formerly, an Act of Congress was required before such a development could be made, but with the enactment of the Federal Water Power Act in 1920 a Federal Power Commission was created, having jurisdiction over all hydro-electric developments in navigable streams. It has always been contended that the lower course of the Susquehanna River was not a navigable stream but, as the Judge Advocate of the War Department decided that it was, it was necessary to secure a Federal license.

It has been pointed out that the dam and power house (the greater part of the investment) is located in Maryland, but that the pool extends into Pennsylvania. This made it necessary to secure the joint approval of the Public Service Commissions of both States. Under the Federal Act the Power Commission takes jurisdiction in all matters upon which the State Commissions are not empowered to rule. In the Conowingo Project, neither of the State Commissions could approve the issuance of securities, and it devolved, therefore, on the Federal Power Commission to make its first official finding in this connection.

When The Philadelphia Electric Company became interested, the first work was to examine all the reports that had previously been made on the development of the power resources of the Susquehanna River. The most capable engineers of the country did some real heroic work in earlier days studying this problem. The late Alfred Noble, Past-President, Am. Soc. C. E., prepared one of the most exhaustive reports on the possibilities of the hydro-electric development on the river. In fact, that report was used as a basis for the final investigations. In the preliminary work, five dam sites, pointed out by Mr. Noble were reconsidered. The final site is not the one recommended in his or in any of the earlier reports. It is known as Site 4A, that is, it is a new site, south of Mr. Noble's Site No. 4.

The rapid advance that has taken place, not only in hydro-electric development and hydro-electric machinery, but in the demand for power, changes all

* Vice-Pres. and Chf. Engr., The Philadelphia Electric Co., Philadelphia, Pa. Dr. Eglin died on February 7, 1928.

the conclusions of the earlier reports, and those that are even a year or two years old are practically out of date to-day (1926). The first one that was made for The Philadelphia Electric Company, contains approximately 230 pages and 50 or 60 plates.

It includes a study of: The water-shed, precipitation, run-off, and the Susquehanna River area flood; ice formation; the quantity of water available for water power development (from official records 1901 to 1927) and the effective head of water that can be secured; the investigation of various sites and locations for hydro-electric plants; the comparison of cost of hydro-electric development at each site; recommendations as to the selection of the most desirable site and a plan for the ultimate development of the total head available; a final analysis and review of the various alternatives possible for the development of Site No. 1 from the economical standpoint, utilizing a maximum of 50 000 cu. ft. per sec. of river discharge; a selection of sites, comparison of costs and possible development; transmission lines and sub-stations, comparing cost of the price of line, line losses, and operating costs; the estimated gross and net output and power deliverable from the hydro-electric plant (operating under probable average conditions) to the sub-station at the end of the transmission line; a complete detail of the operating expenses of the hydro-electric development at each site; a general summary of the project, its operation and operating expenses, utilization, number of units, distribution of loads between the hydro-electric and the steam plant for 1925 to 1928; total capital requirements for the development, including working capital and miscellaneous expenses; forecast of earnings and expenses for the first year of operation and estimates of increase in net earnings from year to year during the first three years of operation; a brief history of the Susquehanna Power Company and other corporations, consolidated with or related to this project; opinions from counsel regarding the status of the development; and the approximate requirements of market within reasonable transmission distance.

After this report was digested the next step was to apply for preliminary permits from the Federal Power Commission and when that application was made, all the material was discussed with the Federal Power authorities, then with the Maryland Public Service Commission, and, finally, with the State of Pennsylvania. As stated, innumerable hearings before public bodies were held. In one report, made to one commission, there were 125 pages of typewritten matter and 9 blue-prints, together with exhibits containing 150 pages and many blue-prints, there being more than 20 blue-prints and about 300 pages of typewritten reports prepared for one commission alone. This illustrates the tremendous amount of preliminary work that was necessary before construction could begin. Many of the engineering conferences started at 9:00 A. M. and ended at 11:00 P. M. The Federal Commission and the Public Service Commissions of both Maryland and Pennsylvania gave their fullest co-operation. It was a long drawn out investigation, but the immensity and importance of the problem made it thoroughly justifiable.

J. V. HOGAN,* ASSOC. M. AM. SOC. C. E.—The work of The Arundel Corporation on the Conowingo Hydro-Electric Project is divided into two contracts. The first is for the construction of the dam from its east end to a point about 50 ft. east of the power house, a length of 3 600 ft. The second contract is for the relocation of the tracks of the Columbia and Port Deposit Branch of the Pennsylvania Railroad, which will be flooded by the construction of the dam. The railroad is to be changed from Port Deposit, Md., to Fishing Creek, Pa., a distance of 16 miles.

Under the Arundel contract 1 200 ft. of the dam comprises the east retaining wall section, and the remaining 2 400 ft. is the spillway section, of which, 1 450 ft. is in the river and must be built in a coffer-dam. The Pennsylvania Railroad tracks parallel the river about 1 100 ft. from the east bank, and the level area on the down-stream side of the dam, between the railroad and the river, affords space for railroad sidings, storage yards, shops, and construction plant. The camp and office buildings are on a narrower level strip between the railroad and the hillside.

Work was started on the dam on March 8, 1926, and the erection of quarters for workmen was begun immediately. The camp has bunk houses, each 20 by 64 ft., built in pairs, connected by a room with showers, lavatories, and flush toilets. Each pair of houses accommodates 56 men in single-deck bunks. The office men are housed in a two-story dormitory containing thirty-six single rooms, each with lavatory and running water. The mess hall has a seating capacity of 600 men. A refrigerating plant, laundry, fumigating house, hospital, and general store serve the camp. The administration building is a two-story frame structure with hollow-tile vault.

A fully equipped machine-shop and a blacksmith shop, for handling heavy forging, have been provided; also a carpenter shop with a platform adjoining it for laying out large form panels. Standard-gauge yard tracks run through all shops. The general storeroom is 50 by 100 ft. Of the standard-gauge railroad tracks 5.8 miles have been laid in the yard, including 1 mile of tracks on elevated trestles.

The water supply is pumped from Octoraro Creek, a distance of 4 000 ft., to a 100 000-gal. storage tank on the hill behind the camp. The water for domestic purposes is filtered and chlorinated, while that for industrial use and for concrete is taken directly from the raw-water storage tank. Hydrants for fire protection are placed on the raw-water mains. Sanitary sewers drain from the camp buildings to two septic tanks. The effluent from the tanks is treated with chlorine before it is discharged into the river.

For the dam it is necessary to remove 170 000 cu. yd. of earth overlying, for a depth of 20 ft., the rock on which the dam is to be founded. This is being done by a 2½-yd., railroad type, steam shovel, with 6-yd., standard-gauge cars and 40-ton steam locomotives. The rock when exposed was seamy and badly disintegrated on top. However, as it was desired to prevent shattering of the foundation, very little blasting could be done, except at places where it was apparent that large quantities of rock would have to be removed. There-

* Secy., The Montrose Constr. Co.; Chf. Engr., The Arundel Corporation, Baltimore, Md.

fore, the rock is being loosened by barring and wedging and by the use of air drills and breakers. Locomotive cranes handle the rock in skips to dump cars, from which it is dumped into the coffer-dam cribs. In places, 30 ft. of faulty rock have been removed. Three motor-driven air compressors, with a total capacity of 2 600 cu. ft. per min., furnish air for this work. Five 25-ton steam locomotive cranes and two gasoline crawler cranes are being used in cleaning up the bottom.

The first coffer-dam was started on April 10, 1926, and was pumped out on July 10. This coffer is 170 ft. wide inside and 1 100 ft. long. The rock-filled cribs are 16 ft. wide, with a 10-ft. opening between them spanned by stop-logs. The water side of the cribs and stop-logs is sheathed with a double layer of 2-in. plank, outside which earth from the excavation is banked. Fig. 1 is a view of the site, and the coffer-dams. The up-stream cribs are carried to Elevation 32.5 and are 30 ft. long, and the down-stream cribs, which are 24 ft. long, to Elevation 28.5. The coffer-dam was built on the rock bottom of the river, which is at an average elevation of +15, by locomotive cranes working ahead on the finished cribs. The opening between this coffer-dam and that on the west side of the river passed a flood of 120 000 sec-ft. without overtopping the cribs. The second coffer-dam was 850 ft. long and extended from the first to connect with the coffer-dam constructed for the power house, and covered all the necessary foundation work to complete this phase of the construction in the second season. The second-year coffer-dam was constructed with alternate crib and stop-log sections similar to the first-year coffer, with the exception that the up-stream coffer-dam cribs were carried to Elevation 42.5, and the down-stream cribs to Elevation 28.5. Up stream the cribs are 20 by 40 ft., and down stream, 16 by 24 ft.

For purposes of stream control during construction, the spillway was concreted in alternate sections above Elevation 20. A section, 52 ft. long, carrying two highway bridge piers was poured, and the adjoining section, 38 ft. long, the clear space between piers, was left open. After these alternate sections were built above high water in the first coffer, the stop-logs were removed, and the river passed through the openings while work in the second coffer-dam was being done. When all the 52-ft. sections have been built to full height, and the highway bridge is completed across the dam, the open sections will be concreted in approximately 5-ft. lifts, the river flow at that time being passed as far as possible through eight 10 by 18-ft. temporary sluiceways constructed in the dam foundation, and arranged so that when the concreting of the alternate spillway sections is completed, these can be closed by means of a drop gate and filled with concrete after the water has risen in the pool to the spillway elevation.

In this work a total of about 430 000 cu. yd. of concrete will be placed. As all the rock from the excavation is needed for weighting the cribs, gravel is being used for the coarse aggregate of the concrete. Some of the gravel is being dredged near the head of Chesapeake Bay, and the remainder shipped from plants near Baltimore, Md. The Bay gravel is towed in scows to Port Deposit, Md., the head of navigation in the Susquehanna, where a derrick

FIG. 1.—VIEW SHOWING TYPE OF CONSTRUCTION USED IN COFFER-DAM OF ALTERNATE CRIB AND STOP-LOG SECTIONS.

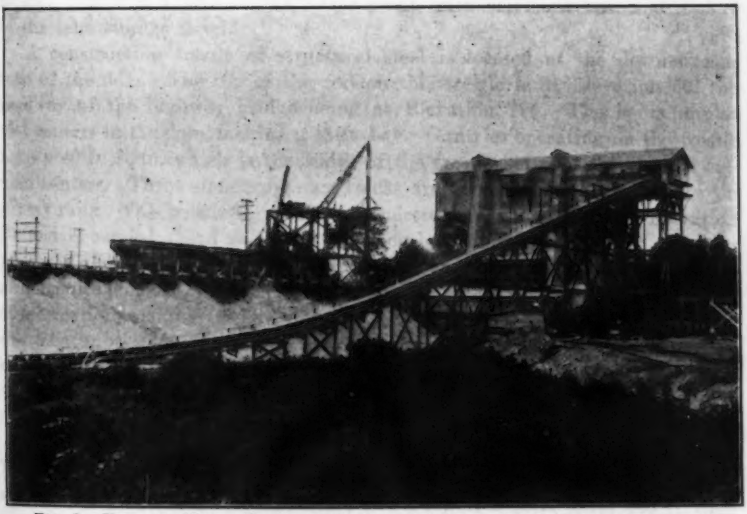

FIG. 2.—VIEW OF MIXER PLANT WITH BELT CONVEYOR, ALSO, SHOWING PART OF SAND STORAGE TRESTLE.

loads it into railroad cars, to be hauled 4 miles to the dam site. Some crushed stone has also been purchased for concrete. The sand is obtained from the same sources as the gravel.

On account of ice in the Upper Bay during the winter, which often prevents towing, a storage of 70 000 cu. yd. of sand and gravel has been provided. This material is stored under a pile bent trestle 20 ft. high, with a total trackage of 2 800 ft., through which it is dumped from 70-ton hopper cars as received from the railroad. A bin of 1 800 cu. yd. capacity, which is also under the trestle, is filled before any material is put in storage. Five 24-in. belt conveyors under this bin deliver on to a 36-in. conveyor inclined 15°, running 450 ft. per min., which feeds the mixing plant. Material from the trestle storage is reclaimed by locomotive cranes which load into cars dumping into the bin mentioned. The 36-in. conveyor delivers on to another shorter belt at the top of the mixing plant, equipped with a tripper for distributing sand and gravel in the hoppers over the mixers.

Cement is received in bulk in box cars, and is unloaded into the boots of two bucket elevators. Screw conveyors at the heads of these elevators distribute the cement into small hoppers over the mixers. When these hoppers are full they overflow into the main cement storage bin, which has a capacity of twenty carloads. This bin, in turn, feeds back to the elevator boots when cement is being taken from it. The mixing is being done by four 2-yd., motor-driven, tilting mixers. The sand and gravel are proportioned by inundators and batchers, respectively, and the cement is weighed on automatic scales. Fig. 2 is a view of the mixer plant, with belt conveyor, and a part of the sand storage trestle.

A construction trestle of structural steel is located at the down-stream face of the dam. The top of the rail on this trestle is at Elevation 60, the roadway of the highway bridge being at Elevation 114. The bents are at pier centers in the dam, making a 45-ft. bay. Gantries operating on the trestle have a 40-ft. span, which is the width of the trestle between the outside column centers. Three standard-gauge tracks are carried on the trestle between gantry rails. This trestle is erected on concrete piers with the top at Elevation 20, and is carried ahead as fast as the rock bottom can be prepared. Except the three main girders which carry the 45-ft. span, and which are riveted plate girders, all members of the trestle are standard rolled shapes with a minimum of fabrication, making for low first cost and high probable salvage. Fifty-four 45-ft. bays, or 2 430 ft. of this trestle, a total weight of 2 400 tons, will be erected west of the present Pennsylvania Railroad tracks. The concrete east of the tracks will be placed by a tower and chutes. Fig. 3 shows the construction bridge with gantry and movable tower.

Three self-propelled gantry travelers operate on the construction trestle. These are steel towers, each of which has a 15-ton stiff-leg derrick mounted with its mast at one up-stream corner, and a concrete hoist tower 100 ft. high near the other up-stream corner. The derricks are all steel, with 80-ft. booms and 2-drum, 100-h.p. electric hoists and attached swingers. The foot-blocks of the derricks are 45 ft. above the trestle floor, or at Elevation 105. These derricks are used for erecting forms and steel, and for setting castings; and,

in case cyclopean concrete should be placed, they would be used for handling plum stones. The concrete towers receive the mixed concrete from cars on the trestle, and elevate it for delivery through chutes to the higher part of the dam. Each tower has a 30-ft. boom section of chute supporting a 30-ft. counterweight chute at its outer end. The gantries have a travel speed along the bridge of 50 ft. per min.

For the lower parts of the work, two chuting outfits suspended from, and moving along, the up-stream trestle girder, are provided for receiving concrete direct from the cars without being elevated. About 25% of total yardage of concrete can be poured through these chutes.

Concrete is hauled from the mixing plant to the towers and chutes by 8-ton gasoline locomotives geared to a speed of 16 miles per hour. Four of these locomotives are now being used, each pulling a 36-ft. standard-gauge flat-car, on which are mounted four 2-yd. side-discharge hoppers. The cars are run directly under the mixers, which are placed so that two mixers may discharge into alternate hoppers on one car simultaneously. The car is then moved up far enough to take the batches from the other two mixers in the remaining hoppers. This plant was designed to mix and place an average of 1 000 cu. yd. per day, working single shift, with a maximum output of 2 500 cu. yd. It has operated very successfully, placing 56 000 cu. yd. of concrete in 1 month, the maximum daily pour to date (October, 1927) being 3 627 cu. yd. in two shifts. A batch from the mixers actually produces 2.22 cu. yd. of concrete in the dam. Wooden forms are being used, made up in large panels which are handled by the traveling derricks. A daily force of about 800 men is employed on the dam construction, including a night shift of about 200 men.

The Columbia and Port Deposit Branch of the Pennsylvania Railroad, at present a single-track road with passing sidings, is to be revised as to alignment and grade for 16 miles. The new roadbed, however, is to be graded for double track the entire length of the change. The relocation follows very closely the 118-ft. contour on the east slope of the river valley down to the dam from where it runs off, on a —0.35% compensated grade, 4 miles to Port Deposit. The hillside is in general very rugged and rocky, and in places so precipitous that the new line approaches the operating line very closely.

As an average of thirty heavy freight trains per day pass over the present road, and as this traffic cannot be interrupted until a single track on the new line is put in operation, it is necessary to shift the present tracks away from the toes of new fills at the points of proximity of the two lines, and to limit blasting to moderate shots practically over the entire line. It is estimated that 1 490 000 cu. yd. of grading will be required, of which, 880 000 has been obtained from cuts and the remainder will be borrowed. In the cuts it developed that 67% was solid rock.

In order to complete the double-track roadbed it became necessary to operate a single track on the relocated line and abandon the old line. This has been done, and under date of October 2, 1927, the first train was operated over the relocated line. This meant a total excavation and borrow of 1 109 000 cu. yd. in 15 months, the start having been made May 25, 1926. This time

FIG. 3.—CONSTRUCTION BRIDGE WITH GANTRY AND MOVABLE TOWER.

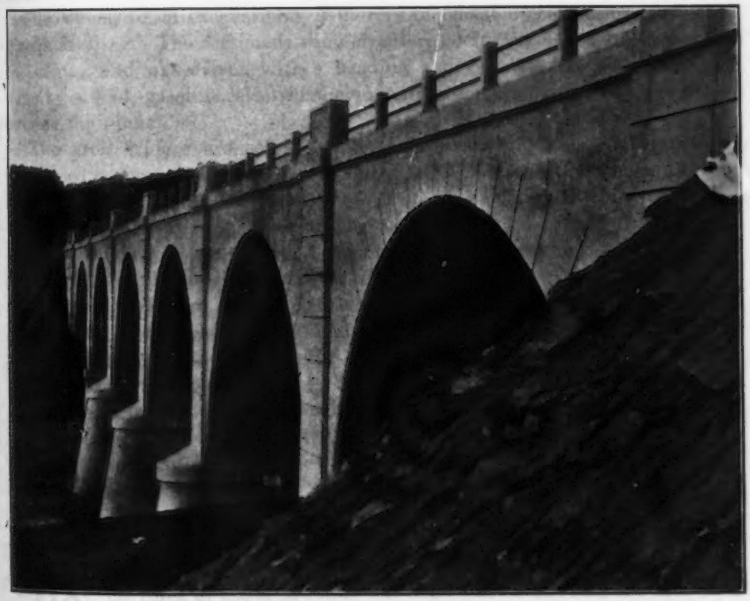

FIG. 4.—TYPE OF REINFORCED CONCRETE ARCH BRIDGE CONSTRUCTION USED ON RAILROAD RELOCATION.

limit, coupled with the difficulties mentioned, required that the grading be attacked at a comparatively large number of points. Nine steam shovels were used on the line. Four of these are standard railroad shovels with 2½-cu. yd. dippers, one is a 2½-yd. shovel mounted on caterpillars, three are full revolving caterpillar shovels with 1¾-cu. yd. dippers, and one a full revolving caterpillar with 1½-cu. yd. dipper. A small revolving shovel is being used for making fills where the present tracks are to be shifted.

Hauling from shovels to dumps is done over a 36-in. gauge track in 5-yd. dump cars. Four 14-ton and seven 8-ton gasoline locomotives and two 16-ton steam locomotives are being used. In several places, tractors and 5-yd. dump wagons made fills. Portable gasoline-driven compressors drive the jack-hammers and tripod drills on the rock work. Nineteen compressors, each of a capacity of 309 cu. ft. per min., are used. A well drill was used in some of the deeper cuts remote from the present railroad where heavy shots could be made. Some of the rock is extremely hard and seams of quartz have been encountered at places, with consequent delays to drilling.

In addition to excavation and embankment, there are three tunnels, three arch bridges, ten arch culverts, and a number of reinforced concrete pipe culverts on the railroad work. Fig. 4, a view of Octoraro Bridge, shows the type of reinforced concrete arch bridge construction on this relocation. The three tunnels are in solid rock and have a total length of 1 020 ft., representing 26 000 cu. yd. of excavation. A battery of portable compressors furnished air for drilling. The section is semi-circular, 30 ft. wide, and 24 ft. high at the center, and was driven with a heading and two benches. Mucking was done by a ¾-yd. gasoline, electric shovel, with the engine-generator set placed outside the tunnel.

The arch bridges are solid barrel, earth-filled, multi-span structures of reinforced concrete. These carry the roadbed over streams at Peach Bottom, Pa., and Conowingo and Octoraro, Md. (Fig. 4), and have three, five, and six 70-ft. spans, respectively. The bridges contain a total of 31 000 cu. yd. of concrete, which was placed with 1-yd. mixers and tower chuting plants. The construction of these bridges was scheduled so that a set of steel centers for three spans were used to complete the three bridges within schedule time. Seven 15-ft. arch culverts, one 20-ft., and one 30-ft. are required on the relocated line. All these, with the exception of the 30-ft. culvert, are now complete and were poured with ½-yd. portable mixers.

The track-laying was completed, using a maximum of four work trains with crews. This was very heavy work, the specifications requiring 130-lb. rail in general, and creosoted ties, together with about 2 miles of track laid with reinforced concrete ties. The ballast was either cinders or slag, and was spread direct from hopper cars. In general, the ties were hand-tamped, and the surfacing was done by track-jacks; but on the concrete ties a Nordberg track-shifter was used, and the track-tamping was done by air tools.

An average force of about 650 men was employed on the railroad relocation work, reaching a maximum of 1 400 during the track-laying period. The men were housed in small camps along the line of the relocation, and in camp cars which were shifted from point to point as occasion demanded.

A. W. CLARK,* ESQ.—The west side of the Susquehanna River, at the site of the Conowingo Hydro-Electric Development, is beautiful rolling farm country. The nearest railroad point, the small Town of Conowingo, is on the east side of the river, on the Columbia and Port Deposit Branch of the Pennsylvania Railroad. The river is spanned at this point by a long and very old highway bridge in poor condition, capable of sustaining a load of not more than 5 tons.

In attacking the work on the west side, it was necessary to build a railroad at the earliest possible moment for transporting materials to the site. A single-track road with an ample number of sidings was constructed from Havre de Grace, Md., on the Philadelphia, Wilmington and Baltimore Division, of the Pennsylvania Railroad, ten miles up the west side of the river, to the dam; 85-lb. rail was used throughout, and the road is ballasted with material secured from the waste bank of a stone-crushing plant, through which the railroad passes. A classification and storage yard has been built about 2 miles down stream from the work. In addition to hauling the freight necessary for construction, a few passenger trains are operated on a schedule planned to meet the working hours of employees who reside south of the dam site and in Havre de Grace. The movements of all trains are controlled by a despatcher near the works.

The first work in the field was the construction of a labor camp, started on March 9, 1926. For more than two months, until the railroad was placed in operation, it was necessary to haul all material to this site by trucks and teams.

A camp with mess halls, commissary store, and hospital has been constructed on the high ground adjacent to the west side of the river. It is capable of taking care of the needs of approximately 1 500 men. A limited number of 2-room and 5-room cottages have also been built for the use of employees with families. The camp streets and all buildings are lighted with electricity, the current for both power and lighting being supplied from the Holtwood Plant of the Pennsylvania Water and Power Company, over a transmission line built from Delta, Pa., about 12 miles distant. The water supply for the entire work is secured from the river, the domestic supply being filtered, chlorinated, and pumped to a 100 000-gal. steel tank, and from there distributed through pipe lines to the camp and the work. A sewerage system is provided with the necessary septic tank, the effluent of which is chlorinated before discharging to the river.

The camp is policed by uniformed guards who are vested with the authority of deputy sheriffs, and a barred room has been set aside as a jail for those who have imbibed too freely and insist on disturbing the peace and quietude of the community.

The area of the river covered by the power plant has been divided into two sections. The section containing the head-works, power house, and a part of the tail-race was first enclosed with a coffer-dam, and while excavation was being done in this section, the construction of the coffer-dam enclos-

* Works Mgr., Div. of Constr. and Eng., Stone & Webster, Inc., Havre de Grace, Md.

ing the remainder of the tail-race could proceed. The first section was completed and unwatered during the last week of July, 1926.

The coffer-dam consists of a double row of rock-filled timber box cribs separated by a sheathed puddle-chamber approximately 10 ft. wide. The cribs were built of commercial lengths of 10 by 12-in. Southern pine and Douglas fir and were tied together across the top of the puddle chamber with the same material, securely bolted in place. The sheathing of the clay chamber was a double course of 2 by 10-in. planking, with lapped joints.

The cribs were built up in place and carefully fitted to the bed of the river, with the bottom bearing members placed lengthwise in the direction of pressure. Caterpillar cranes, that traveled on standard-gauge tracks along the top of the completed cribs, were used to unload timbers from cars. The same tracks were also used for supplying the rock-fill and material for the clay puddle which was dumped from 6-yd. and 12-yd., two-way, dump cars.

The following quantities give an idea of the magnitude of the work necessary for completing the first section of the coffer-dam which encloses the head-works, power house, and a part of the tail-race, and comprises an area of 14.7 acres: 2 575 lin. ft. of coffer-dam; 5 600 000 board ft. of timber; 69 000 cu. yd. of rock-fill; and 32 000 cu. yd. of clay.

The second section of the coffer-dam which encloses the remainder of the area covered by the tail-race, 11.1 acres, was completed the first week in June, 1927, and contains the following quantities: 2 220 lin. ft. of coffer-dam; 2 700 000 board ft. of timber; 24 000 cu. yd. of rock-fill; and 13 200 cu. yd. of clay.

In planning the excavations for the first section, it was decided to concentrate on the head-works and power house, so that they could be constructed while the excavation for the tail-race was being done. The first-mentioned work was completed the last week in April, 1927, and consisted of the removal of approximately 100 000 cu. yd. of rock. The total quantity of rock removed for the tail-race was approximately 200 000 cu. yd.

The excavation for the head-works and power house was carried down to several different elevations, the deepest and most difficult being in the area containing the draft-tubes. This consisted in cutting a gash in the river bed approximately 550 ft. long, 80 ft. wide, and 30 ft. deep, with the up-stream side vertical, and the down-stream side sloping up in the tail-race at a rate of 1 on 6. For this particular work, a high-lift, full-revolving steam shovel on caterpillars, with a 2.5-yd. bucket, was selected. Its use minimized the number of track changes and also made it possible to remove a large quantity of material with the hauling tracks on the bed of the river or on the top of the cut. For the remainder of the work, a 3-cu. yd. steam shovel, first operated on railroad trucks and, later, mounted on caterpillar trucks, was used where the lifts were low, and two small revolving shovels on caterpillars were used for odds and ends. All this equipment was served with 6 and 12-cu. yd., two-way dump cars, hauled by steam locomotives on a standard-gauge track system. A lead connected the construction track to double-track trestle, running parallel to the down-stream leg of the coffer-dam on a 2.5% ascending grade.

Four 800-ft. compressors located on a side hill well above the possibility of flood, supplied air for the rock drills. The rock from the lower tail-race coffer-dam was dumped on an island which divides the tail-race into two channels. The dump was made from a track on a 3% grade laid on trestle and fill.

Part of the rock in the excavations was suitable for concrete and was crushed and used. A crushing plant was installed on the west bank about 1 500 ft. down stream from the dam. Stone was carried from the tertiary crushers on a belt conveyor to a 25 000-cu. yd. storage pile under which, at sufficient intervals, there were bin gates for delivering the crushed stone to a belt conveyor operating in a 6 by 6-ft. reclamation tunnel under the entire length of the pile and then up an incline to the stone bins over the concrete mixing plant. Around the bin gates were steam coils to deliver live steam to the material in winter.

Sand was delivered to the job in standard hopper bottom cars hauled over the construction railroad from Havre de Grace, and dumped from a trestle to a storage pile under which there is a 6 by 6-ft. tunnel with conveying equipment for carrying the sand to the mixing plant similar to that used on the stone.

The concrete mixing equipment consisted of two 2-cu. yd. motor-driven mixers supplied with aggregates delivered into the mixers from batchers and inundators which were filled from overhead bins. The proper proportion of cement was determined by weight. Bulk cement was stored in wood silos by means of bucket elevators operating from the track level on the ground. Wherever it was feasible, steel supports were used in the construction of the concrete plant in order to reduce the fire hazard.

Concrete was hauled from the mixing plant by 8-ton gasoline locomotives in 2-cu. yd. standard gauge, specially designed tilting cars, of the "roll-over" type. Four wooden towers, approximately 220 ft. high, supported the necessary chuting equipment.

For repairs to construction equipment a shop group was established about ½ mile down stream from the dam. A machine shop, blacksmith shop, pipe shop, and repair pits, equipped with an air hammer, lathes, grinders, drill presses, and other machine tools for the handling of such work, was built. As far as possible, equipment was purchased for these shops, that could be installed in the permanent shops.

A carpenter shop was constructed, in which the necessary wood-working machinery to facilitate the construction of forms, was installed. Space was reserved for lumber storage adjacent to the shop.

A place was also reserved in this vicinity for the storage of reinforcing steel, and, here, a power shear and power bender were installed for the fabrication of reinforcing steel. All these shops and the steel yard were served with standard-gauge tracks, which connected with the construction railroad and the track system throughout the work.

The power station is of structural steel and concrete. The work was arranged so that the concrete necessary to support the steel frame was placed as early as possible, in order that the structural steel could be erected and

traveling cranes put in operation to handle the installation of the heavy equipment.

The draft-tube concrete was the first step in this program, and concrete was completed in the elevation required for setting the draft-tube liners during the first week in June, 1927. As this work progressed, it was followed closely by concrete walls, carried to the main floor level, which allowed the erection of structural steel to start in the early part of June, and permitted the operation of the main building crane by the end of July.

By making use of the five 10-ton derricks which served the concrete operation, the erection of the water-wheel equipment was started during the latter part of April. In this way, six of the seven units were completed as far as the erection and riveting of the scroll cases and pit liners by the time the building cranes were available for the heavy lifts of the butterfly valves. As fast as the valves were in place and riveted, the concrete fill around the scroll cases was poured, and the interior of the generator room completed.

H. A. HAGEMAN,* M. AM. SOC. C. E.—Mr. Wilson has given the general features of the hydraulic design of the Conowingo Project. It is the purpose of this discussion to set forth in somewhat greater detail the design of the major parts of the structure and operating equipment.

Station Intake.—On account of the length of waterway required to accelerate the water and the width of the space necessary to house the electrical bay, the intake is in itself a stable structure. It will serve adequately as a dam and no support from the power station substructure is necessary. For the four future units, this makes necessary, at present, the construction of the intake only, and results in a minimum initial expenditure. It will also make possible the future realization of all progress in water-wheel design which may be made prior to the installation of these units.

Trash Racks.—As Mr. Wilson has explained, the entrances to the waterways of the turbines will be located near the bottom of the reservoir and a curtain-wall extending 40 ft. below the normal pond surface will be constructed up stream from them as a protection against ice. The curtain-wall is a distinct feature of the intake and is more economical than any other form of ice protection of equal efficiency. Each waterway will be divided into two parts from the entrance to a point a short distance up stream from the butterfly valve, where they will be merged into one.

The trash racks are located in each intake opening a sufficient distance behind the ice-protection wall to permit the operation of a mechanical rack rake. These racks will be made up in panels of convenient size, which will slide in guides set in the concrete. The lower panels in each opening will be equipped with flat steel bars, but the upper panels, which are located behind the curtain-wall and are, therefore, of no use in providing actual waterway area, will be faced with treated timber. The timber facing on the panels will decrease the adherence of ice to the racks at the water surface and also reduce the cost of the panels.

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The rack sections have been designed for an unbalanced head of 5 ft. over their entire area, under which conditions the maximum stress in the steel will not exceed 16 000 lb. per sq. in.

The guides for the emergency sectional head-gates will be located down stream from the trash racks. These gates will be of structural steel and will be used to close the waterways when it is desired to unwater them in order to inspect or repair the butterfly valves. The valves are used normally as head-gates.

A sufficient number of gate sections will be provided to close the intake of one unit. The top, or sealing section, will be arranged to seal against a casting at the top of the intake opening so that it will not be necessary to extend these gates to the water surface.

Steel stop-logs of similar design to the head-gates, but with sections of less height, will be provided to close the waterway openings for the future units until such a time as these units are installed.

Gantry Cranes.—The sectional head-gates and trash racks will be handled by the gantry cranes which travel along the intake and spillway sections of the dam. For this purpose the cranes will be equipped with a special lifting beam which will run in the guides for either the head-gates or trash racks. This lifting beam will attach automatically to a rack panel or head-gate section under water, so that each section may be hoisted out of the guides separately and transported to a suitable storage space by the cranes. When it is desired to place the head-gate, trash-rack, or stop-log sections in the intake the process will be reversed, the lifting beam automatically releasing the sections when they have reached their proper place.

The wide bellmouth provided for the waterways insures very satisfactory hydraulic conditions for the intake with the head loss at the entrance reduced to a minimum.

Velocity at Intake.—With the water-wheels discharging 5 300 cu. ft. per sec. corresponding to full generator load at 89-ft. head, the velocity through the trash racks will be about 2.6 ft. per sec. The velocity at the entrance of the waterways will be 2.8 ft. per sec. In the distance of 71 ft. from the entrance of the waterways to the center line of the butterfly valves the water will be accelerated to a velocity of about 12 ft. per sec. After passing the center line of the butterfly valve, the velocity decreases and the water enters the plate-steel scroll case of the water-wheels at about 9.3 ft. per sec.

Butterfly Valves.—The area of the entrance to the water-wheel intakes for each unit is about 1 900 sq. ft. To close this area with two or even three lift-gates of any developed type would require gates of enormous size for the pressure to be resisted, with the consequent difficulty of obtaining an adequate seal against leakage. The operating mechanism required would be expensive and the operation slow.

By placing a circular gate or butterfly valve at the entrance to the scroll case, the area to be closed is only about 570 sq. ft. as compared to the 1 900 sq. ft. for rectangular gates. Further, the perimeter to be made water-tight is only about 85 ft. for the circular gate compared to 350 ft. for the rectangular gate.

Butterfly valves, 27 ft. in diameter, of the vertical shaft, circular type, were selected as being the most practicable to suit the conditions. They will be designed to open or close in about 5 min. and may be closed with maximum flow through the turbine. In the closed position they will stand at right angles to the line of flow. Each valve will be sealed after closing by admitting water pressure to a rubber tube, 3 in. in diameter, set in the inside face of the valve housing opposite the periphery of the valve wicket. Based on tests of this sealing device, the leakage through these enormous valves will be negligible. Each valve will be operated by oil pressure in a cylinder 40 in. in diameter.

These valves will be the largest ever built, the largest now (1926) in existence being 23 ft. 6 in. in diameter, built by the Allis-Chalmers Manufacturing Company for the Niagara Falls Power Company. For the Conowingo development, three valves are being built by The Wm. Cramp and Sons Ship and Engine Building Company and four by the Allis-Chalmers Manufacturing Company, each manufacturer supplying the valves to serve as the head-gate for the water-wheels he is to furnish. The sealing device will be the same for all valves and the manufacturers have co-operated in the design of bearings, operating mechanism, and renewable parts, making them alike and interchangeable.

Water-Wheels.—The Allis-Chalmers Company and The Wm. Cramp Company are co-operating in the design of the water-wheels, with the result that only one set of principal spare parts will be required. The outward appearance of the units will also be similar. To this end, such parts as lignum vitæ guide-bearings are of the same design for all wheels and are interchangeable. The runners, while not of the same design, will have the same limiting dimensions, so that the spare runner which will be supplied by The Wm. Cramp and Sons Ship and Engine Building Company, will serve in either a Cramp or Allis-Chalmers setting with high efficiency.

It is expected that the main water-wheel units will have a maximum efficiency of at least 93% and also that, in an average year of run-off, the seven main generators will have an energy output of 1 382 000 000 kw-hr. Plate-steel scroll cases were chosen on account of their greater economy when compared to concrete which would require heavy reinforcing in the substructure and much more elaborate and expensive form work. The maximum diameter of the runner will be 17 ft. 9 in., and its weight, when completely assembled, will be about 240 000 lb. Each main shaft will be of forged steel about 36 in. in diameter, with top and bottom flanges forged integrally with it.

A structural steel pit liner and generator support will be provided for each main unit. It will be attached to the speed ring of the water-wheel and be designed to carry the weight of the generator, thrust-bearing load, and adjacent floor loads down to the speed ring, whence the load is transferred through the speed-ring vanes to the power station substructure. The support will be built of plate and structural steel instead of cast iron, as is ordinarily the case. The support is essential to the power station design as it was found that it would be very difficult to design a concrete foundation for the generator in the space available to properly transfer the enormous load to the speed ring through which it must be transmitted to the substructure foundation.

Governors.—Each water-wheel manufacturer will supply with the unit which he furnishes, actuator type, oil-pressure, relay governors of his own make, having governor flyballs mechanically driven from the main shaft of the water-wheel.

Power House.—The units and auxiliaries will be arranged in pairs. Units Nos. 1, 2, 5, and 6 will have Allis-Chalmers turbines and General Electric generators. Units Nos. 3, 4, and 7 will have Cramp turbines and Westinghouse generators.

Each pair of units will be equipped with an interconnected oil-pressure system consisting of one pressure tank and duplicate pumps and sump tanks. The governor actuators and pressure tanks will be located on the main floor of the station at Elevation 46.0 ft., but the pumps and sump tanks will be located on the station auxiliary floor at Elevation 35.0 ft.

A central oil-purification system will be provided with the necessary piping to drain any governor system and purify the oil and return it to the system.

Draft-Tubes.—The draft-tubes for each make of water-wheel will be built in accordance with the design of their respective manufacturer. The Cramp units will have tubes of the conventional Moody type with a concrete cone extending from the bottom at Elevation — 14.5 ft. to the hub of the runner. The Allis-Chalmers units will be equipped with a White Hydraucone type of draft-tube and will have its bottom at Elevation — 18.5 ft. The bottom of the outlet of the draft-tube will be at the same elevation for all units.

The substructure over the draft-tube of each unit will be supported by five cast-steel stay vanes which each water-wheel manufacturer is furnishing for the units he supplies. Down stream from the circle of stay vanes, the draft-tube will be divided into two parts by a central bearing wall.

The low setting of the water-wheels relative to the tail-water level, required for the protection of the runners against pitting, makes it necessary to provide for stop-logs at the outlet of the draft-tubes to prevent the unit from being flooded by tail-water when unwatered for inspection or repair. These stop-logs will be handled by a small gantry crane in a manner similar to the sectional head-gates and trash racks.

All waterways will be arranged with suitable connections to a drainage tunnel which will be located under the inspection tunnel, up stream from the draft-tubes. This drain will empty into a sump at the west end of the station from which the water will be pumped into the tail-race.

Spillway.—As already explained by Mr. Wilson, the head-water level at the station is to be maintained constantly at Elevation 108.5 ft. except during periods of extreme drought, at which time the draft from the reservoir will cause the head-water to fall below the normal elevation. In order to pass flood flows in excess of the requirements of the water-wheels a spillway equipped with crest gates will be provided.

The spillway is of the conventional ogee type, having a parabolic crest and a circular curved surface at the toe. The piers supporting the crest gates are 7 ft. thick and 45 ft. on centers, leaving a clear opening of 38 ft. between them. These piers also support the highway and the runway for the gantry cranes that operate the gates.

Crest Gates.—The crest gates will be of the Stoney type, 22 ft. 6 in. high by 40 ft. wide, with timber seals at the bottom and rubber-covered staunching rod seals at the sides. Fifty crest gates will be provided, having a combined discharge capacity of about 800 000 cu. ft. per sec., with the head-water level at 108.5 ft. Three smaller regulating gates, each having a capacity of about one-third that of a crest gate, will be provided for the finer adjustments of flow. Up stream from the crest gates, additional guides will be set in the piers to receive a service gate. This gate will be of the fixed roller type and will be used to close the gate-opening when it is necessary to remove one of the crest gates or regulating gates for painting or repairs.

The crest gates, regulating gates, and service gates will all be handled by the same cranes that handle the sectional head-gates. A total of three gantry cranes will be provided for handling gates on the dam and the sectional head-gates and trash racks. For handling the crest gates they are equipped with special lifting beams of different design from that used to handle the sectional head-gates and stop-logs. As this lifting beam is never submerged, it is arranged for an operator to ride on it and operate the locking device which attaches it to the gates. When the gates are raised to their open position they will be held in place by means of pawls engaging suitable stops on the ends of the gates, thus permitting the crane to move on to the next gate.

A study of the duty cycle of these cranes indicates that one crane will lift a crest gate every 9 min. As the maximum rate of rise of any flood on record has been such as to require the opening of one gate only every 30 min., ample factor of safety has been provided in these gantry cranes to insure the timely opening of these gates.

ALEXANDER WILSON, 3D,* ESQ. (by letter).—While the schedule called for operation of the first unit about July 1, 1928, construction progress was so much better that all the units were in operation by that time, the first two units having been placed in commercial operation on March 1, 1928.

In the low-water season of 1926, a coffer-dam of the puddle-chamber type was constructed, enclosing the power house and west branch of the tail-race, an area of about 14 acres. The top of this coffer-dam was built to Elevation 42, where it was estimated that it would not be topped by a flood of any volume less than 350 000 cu. ft. per sec., so that work on the heavy excavation of the power-house foundations and draft-tubes could reasonably be expected to proceed without interruption by floods in the spring of 1927.

Simultaneously with the construction of the power-house coffer-dam, another coffer-dam of the spaced crib type was built from the Cecil County shore, extending to within approximately 700 ft. of the power-house coffer-dam. About the middle of November, a flood of 350 000 sec-ft. occurred, the greatest November flood that has been recorded for the Susquehanna River. The head-water elevation was raised to within a few inches of the top of the sand-bags which had been placed on the power-house coffer-dam and, of course, topped the spaced crib coffer-dam on the other side of the river. As the foundations for the dam had been completed in this area with alternate spillway sections, each including two

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bridge piers constructed to a height above the river level, it was not necessary again to unwater this coffer-dam after the flood had subsided.

During the winter and early spring of 1927, the pouring of the pier sections was completed up to and including the highway bridge, leaving intermediate sections 38 ft. wide at the level of the foundation (Elevation 22) to provide for the diversion of the river during the summer of 1927. Likewise, the head-works at the power station were completed during this period up to an elevation well above flood level.

As soon as the spring floods of 1927 had subsided sufficiently, the coffer-dam was extended from the last completed spillway section to the power-house head-works, and, by September, the foundations of the dam had been completed in this last coffer-dam and the alternate sections poured above the elevation of the top. With the removal of this coffer-dam, twenty 38-ft. notches in the spillway, together with eight 10 by 18-ft. sluices through the spillway, were taking care of the river flow.

The final closure of the notches was commenced early in October, 1927. The easterly ten notches were first brought up to Elevation 30, and the west ten to Elevation 35. These two groups, with a 5-ft. difference in bottom grades, were then raised alternately in 10-ft. lifts, thus providing a flow of 5 ft. through ten openings, in addition to the flow through the eight sluices. In the actual closing of the waterway at each 38-ft. opening, a steel and timber flap-gate, hinged at the top from the adjoining spillway sections, was used. The gantry cranes provided for the crest gates were used to handle these flap-gates. With the raising of the notches and the corresponding rise of the head-water level, the discharge capacity of the eight sluices increased so that finally, when the notches had been raised to about Elevation 50, the discharge from the sluices was equal to the river flow and the closing of the notches was completed without the use of flap-gates.

After the concrete in the notches had been completed to crest elevation, final closure of the eight sluices was made by lowering specially designed, reinforced concrete gates, weighing 75 tons each, and then filling the sluice-way opening with concrete.

Closure of the notches was hampered through November and December, 1927, by high water. Favorable flow conditions occurred in January, 1928, however, permitting completion to crest elevation on January 16 and, on January 17, 1928, the sluice-way gates were dropped, making the final closure of the dam.

The Columbia and Port Deposit Branch of the Pennsylvania Railroad was put in operation on the relocated line on October 10, 1927, just in time to escape a heavy flood which washed out part of the old line above the dam.

The highway bridge over the dam was opened to the public on November 15, 1927, and two weeks later the old Conowingo Bridge was destroyed by dynamiting the superstructure and dropping the spans to the bottom of the river.

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Paper No. 1711

PRECISE WEIR MEASUREMENTS*

By Ernest W. Schoder,† M. Am. Soc. C. E., and the Late Kenneth B. Turner,‡ Esq.

With Discussion by Messrs. H. W. King, Floyd A. Nagler, A. Streiff, R. L. Parshall, W. S. Pardoe, R. E. Ballester, Gardner S. Williams, Th. Rehbock, Erik G. W. Lindquist, Clemens Herschel, and Ernest W. Schoder.

Synopsis

This paper presents the results of extensive new volumetric measurements of the discharge over weirs of the sharp-crested (or nearly sharp) type, occupying the full width of the channel (that is, without end contractions). In addition to experimental data by the writers, the paper includes also new experimental data by Charles L. Walker, M. Am. Soc. C. E., and Carl R. Weidner, Assoc. M. Am. Soc. C. E., William F. Martin, Francis M. Dawson, M. Am. Soc. C. E., Henry R. J. Meyer, Assoc. M. Am. Soc. C. E., and Russell A. See, M. Am. Soc. C. E., Jacob O. Jones, Assoc. M. Am. Soc. C. E., and Charles M. Weber, Assoc. M. Am. Soc. C. E.

General.—The heights of the weirs ranged from 0.5 to 7.5 ft., the heads, from 0.012 to 2.75 ft., and the widths of channel, from 0.9 to 4.2 ft. Included in the paper are the results of 2 438 separate volumetric measurements of discharge for 1 512 different heads, all made at Cornell University and now published for the first time. Of these, 1 276 measurements for 530 heads were

Note.—For Table of Contents, see p. 1109.

* Published in September, 1927, *Proceedings*.

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‡ Mr. Turner, formerly Assistant Professor of Hydraulics, School of Civil Engineering, Cornell University, died October 21, 1918. The paper is presented by Professor Schoder. The writers jointly had completed all the experimental work, had made the primary calculations, and had reached the main conclusions in a general way, although not as far as many refinements and important comparisons are concerned. Only the first of the formulas involving r_a (see p. 1038) had been suggested or tested.

TABLE 1.—CONDENSED SUMMARY OF WEIR EXPERIMENTS.

(All Weirs Full Width of Channel (Without End Contractions). Every Discharge Measurement Volumetric.)

Series.	Date.	Height of weir, in feet.	Length of weir, in feet.	Approximate range of heads, in feet.	Number of separate volumetric measurements.	Number of different heads.	Number of runs with velocity distribution measured.	Remarks.
Schoder and Turner A	Feb. 2-Mar. 11, 1911	7.53	4.22	0.05 to 2.44	63	27	10	Old iron plate.
" " B	Mar. 25-31, 1911	5.51	4.22	0.05 to 2.44	69	26	11	New iron plate.
" " C	Feb. 8-Apr. 12, 1914	5.84	4.22	0.036 to 2.53	127	60	7	Part of old crest from 16-ft. canal.
" " D	Dec. 27, 1914-Jan. 2, 1915	7.50	4.22	0.028 to 2.75	111	40	16	
" " E	Jan. 4-9, 1915	0.50	4.22	0.029 to 2.00	57	29	11	
" " F	Jan. 10-15, 1915	1.00	4.22	0.029 to 2.36	91	37	17	
" " G	Jan. 21-28, 1915	1.50	4.22	0.018 to 2.41	90	37	15	
" " H	Jan. 30-Feb. 4, 1915	2.00	4.22	0.022 to 2.47	87	34	14	
" " I	Feb. 5-7, 1915	3.00	4.22	0.022 to 2.67	98	36	15	
" " J	Feb. 8-11, 1915	4.00	4.22	0.017 to 2.70	93	36	15	12-in. rolled brass plate carefully maintained.
" " K	Feb. 12-24, 1915	5.50	4.22	0.013 to 2.65	100	38	16	
" " L	Feb. 28-Mar. 1, 1915	7.50	4.22	0.018 to 2.24	37	24	3	
" " M	Feb. 26-Mar. 4, 1916	7.50	4.22	0.014 to 2.53	72	31	9	
" " N	Mar. 26-Apr. 16, 1916	0.75	4.22	0.012 to 2.33	100	42	23	
" " O	Apr. 17-23, 1916	5.50	4.22	0.05 to 2.71	74	29	10	
" " P	Jan. 13-18, 1920	2.50	4.22	1.01 and 2.04	12	4	4	
Dawson I	Jan., 1913	5.51	4.22	0.10 to 2.54	39	18	...	Schoder and Turner B after 2 years.
" II	May, 1913	5.49	4.22	0.11 to 2.65	53	16	...	New sharp.
" III	June, 1913	5.49	4.22	0.11 to 2.08	33	10	...	Crest II after 6 weeks.
Martin	Jan., 1906	1.20	2.023	0.095 to 1.064	30	30	...	New crest.
Meyer and See 1 A	Mar. 26-Apr. 2, 1914	2.50	2.02	0.05 to 1.34	106	63	...	Brass crest.
" " 1 B	May 12, 1914	2.287	2.02	0.04 to 1.32	38	38	...	" "
" " 1 C	May 14, 1914	2.068	2.02	0.05 to 1.33	37	37	3	" "
" " 1 D	May 16, 1914	1.742	2.02	0.05 to 1.32	37	37	...	" "
" " 1 E	May 16-18, 1914	1.409	2.02	0.05 to 1.30	38	38	3	" "
" " 1 F	May 19, 1914	0.975	2.02	0.04 to 1.28	38	38	...	" "
" " 1 G	May 19-21, 1914	0.755	2.02	0.05 to 1.25	37	37	3	" "
" " 1 I	May 28, 1914	0.000	2.02	0.10 to 0.95	36	36	...	" "
" " 2 B	Apr. 17, 1914	0.198	2.02	0.05 to 1.12	36	36	...	" "
" " 2 C	Apr. 18, 1914	0.408	2.02	0.044 to 1.17	38	38	...	" "
" " 2 D	Apr. 25, 1914	0.727	2.02	0.052 to 1.26	38	38	...	" "
" " 2 E	May 2, 1914	1.003	2.02	0.047 to 1.29	37	37	...	" "
" " 2 F	May 5, 1914	1.409	2.02	0.046 to 1.31	37	37	...	" "
" " 2 G	May 7, 1914	1.807	2.02	0.052 to 1.30	38	38	...	" "

TABLE 1.—(*Continued*).

Series.	Date.	Height of weir, in feet.	Length of weir, in feet.	Approximate range of heads, in feet.	Number of separate volumetric measurements.	Number of different heads.	Number of runs with velocity distribution measured.	Remarks.
J. O. Jones A	Apr., 1915	2.50	2.018	0.0196 to 1.339	42	32	3	
" " B	" "	2.50	2.018	0.02 to 1.357	31	28	...	
" C	" "	2.50	2.018	0.019 to 1.357	32	28	3	
" D	" "	2.50	2.018	0.016 to 1.355	36	29	3	
" E	" "	2.50	2.018	0.020 to 1.354	29	27	3	
" F	" "	2.50	2.018	0.021 to 1.353	29	27	3	
" G	" "	2.50	2.018	0.020 to 1.353	29	27	3	
" H	" "	2.50	2.018	0.017 to 1.356	38	29	3	
C. M. Weber II	May and June, 1915.	2.50	2.018	0.021 to 1.356	37	32	3	
" III	" " "	2.50	2.018	0.025 to 1.358	34	29	3	
" IV	" " "	2.50	2.018	0.018 to 1.355	30	27	3	
" V	" " "	2.50	2.018	0.017 to 1.353	29	26	3	
" VI	" " "	2.50	2.018	0.089 to 1.363	20	19	3	
Total number volumetric measurements made at Cornell University now published for the first time					2 438	1 512	241	
THE FOLLOWING DATA COVER THE CLASSIC EXPERIMENTS USUALLY QUOTED IN AMERICA.								
Fteley and Stearns*	Mar., 1877	3.17	5.00	0.07 to 0.82	24	24	...	Omitting doubtful runs.
" "	Nov.-Dec., 1879	6.55	19.00	0.47 to 1.603	9	9	1	
Bazin† Series I	Sept.-Oct., 1886	3.72	6.56	0.19 to 1.012	67	67	...	
" " II	Oct., 1886	3.72	3.28	0.187 to 1.338	38	38	...	
" " III	Nov.-Dec., 1886	3.297	1.64	0.19 to 1.78	48	48	...	
J. B. Francis	Oct.-Nov., 1852	5.048	9.997	0.62 to 1.55	46	5	...	Two end contractions.
" "	Nov., 1852	5.048	7.997	1.018	2	1	...	Four end contractions.
" "	Oct.-Nov., 1852	2.014	9.997	0.65 to 1.05	19	3	...	Two end contractions.
" "	Nov., 1852	5.048	9.995	0.795 and 0.979	12	2	...	Expansions down stream.
" "	Nov., 1852	5.048	9.995	1.00	5	1	...	No expansions nor contractions.
" "	Oct.-Nov., 1852	2.014	7.997	0.680	4	1	...	Four end contractions.

* Fteley and Stearns in addition to their volumetric measurements ran 94 experiments using comparative discharges (not volumetric) with 21 heads on weirs of various heights above the bottom of the channel of approach, which were 0.5, 1.00, 1.70, 2.60, and 3.56 ft. These were without end contractions. In addition, 5 heads, 18 experiments were run on weirs with end contractions.

†In addition, Bazin Series IV to X, 228 experiments were made with comparative discharges as calculated from his standard weir which was calibrated by his Series I, II, and III.

made by the writers. The distribution of the velocities in the channel of approach was measured by current meter for 241 heads (196 by the writers), including many cases with abnormally high velocities near the surface and near the bottom. The experiments were made at various times between 1904 and 1920, with the major part between 1913 and 1916. A condensed summary of the experiments is given in Table 1.

Findings.—The main findings are:

(1) Considerable limitation should be understood as to the widespread idea that a precision in discharge estimates within 1% or 2% is obtainable merely (*a*) by careful observations of the head of water on a weir having nominally a sharp, square edge, but built without precise regard to the degree of sharpness and smoothness of the crest plate and to the velocity conditions in the channel of approach; and (*b*) by the use of the standard formulas or other estimating devices of present engineering practice.

(2) The use of the simple Francis weir formula:

$$Q = 3.33\ L\ h^{\frac{3}{2}}, \text{ or } Q = 0.623 \times \frac{2}{3}\ L\ h \sqrt{2\,g\,h}$$

as a basic formula is substantiated for cases closely approximating the ideal weir with a truly sharp, square edge, with smooth, vertical, up-stream face near the crest, and with a deep up-stream pool, or with negligible effect of velocity of approach. (For low heads, see Item (5)).

(3) This paper shows the extent and nature of the inadequacy, for general precise work, of formulas that introduce merely the mean velocity of approach; or that, as the equivalent, introduce merely the height of weir; also the inadequacy of curves and tables that merely average or approximate the data of the experiments hitherto published. This inadequacy consists chiefly in the failure of the experimenters to measure and place on record the distributions of velocities in the channel of approach. The mere statement that velocities were approximately equalized by baffles leaves the matter too uncertain except for a limited range of heads and heights.

(4) The new experimental data (by several experimenters) agree with those of Francis, Fteley and Stearns, and Rehbock, in their evidence that Bazin's coefficients and formula for sharp-crested weirs give too high discharges for low heads—by fully 2 to 3% for heads of 0.3 ft. to 1.2 ft.

(5) For low heads of less than about 0.4 ft. for weirs more than 2 ft. high: (*a*) an additive corrective expression of the Fteley and Stearns' type, but made to the simple Francis formula, $Q = 3.33\ L\ h^{\frac{3}{2}} + 0.007\ L$, is verified, although their value of 0.007 ft-sec. units is to be replaced by other values ranging from 0.004 to 0.012 in certain cases; (*b*) many data of percentage corrections for extremely low heads (less than 0.10 ft.) on a variety of crest pieces are graphically shown.

(6) The percentage increase in discharge (as compared with sharp-crested conditions) due to a slight rounding of the up-stream top corner of the crest (say, to as much as $\frac{1}{24}$-in. radius = about 1 mm., which may soon occur with exposed steel and which has been overlooked or neglected in cases intended to be "sharp-crested") is shown to be as much as 2% for heads of 0.50 ft. and 0.5% for heads of 1.35 ft. A rounding of $\frac{1}{8}$-in. radius causes

about 3%, and one of ¼-in. radius, about 5½%, increased discharge at 0.50-ft. head. Agreement is found with Fteley and Stearns' data for the larger roundings.

(7) The percentage increase in discharge due to changing the roughness of the up-stream face of the weir bulkhead from that of a polished brass plate to that of a coarse file for a distance of 12 in. below the crest is shown to range from about 2% for 0.50-ft. head to about 1% for 1.35-ft. head. This agrees with the previous findings of James Barr for V-notched weirs.

(8) The new data show no justification for a taboo on a weir with a head of water as great as, or greater than, the height of the crest above the bottom of the channel. When the use of such a weir is convenient, the new formula, involving auxiliary velocity measurements, and several simpler formulas are amply accurate. In fact, there seems to be an inherent certainty of action for the very high heads on very low weirs.

(9) While the importance of careful measurements of heads, including the precise referencing of the gauge to crest level (zero determination), and the use of appropriate instruments, are not to be minimized, yet all the writers' work substantiates the conclusions of earlier experimenters that there is no necessity for slavish imitations in the precise manner of determining the head, nor for a precise location of station up stream from the weir to agree with a particular experimenter, provided no violence is done to the general hydraulic circumstances. Hitherto, many errors have been charged to insignificant details of the head determination when the trouble has been due to lack of appreciation of other factors and to neglect in making proper arrangements or measurements.

As a general rule, rarely subject to exceptions, the head should be determined in one of two ways: (1) by a direct measurement in the channel of approach (by one of several appropriate devices); or (2) by a piezometer with a stilling-well such that no correction is necessary.

(10) For precision it is necessary to deal not (as in most formulas) with the head corresponding to the mean velocity of approach, but with the head corresponding to the velocity above the level of the weir crest. This involves auxiliary measurements by current meter, Pitot tube, or floats, etc.

Where it is possible actually to duplicate the definitely known velocity conditions prevailing for different heads and heights in the channel of approach, the use of a special formula or special coefficients fitting the particular case remains entirely proper.

The extreme difference in discharge due to altering the distribution of velocities up stream from a weir, keeping the head constant, amounted to 26 per cent. The auxiliary velocity measurements by the writers and the introduction of this new factor are, in fact, simply carrying to a logical conclusion the suggestions of J. B. Francis, Hamilton Smith, Fteley and Stearns, Bazin, and others.

(11) A tentative formula,

$$Q = 3.33\, L \left[\left(h + \frac{v_a^2}{2g} \right)^{\frac{3}{2}} + \frac{h}{3.33} \times \frac{v_b^2}{2g} \right]$$

is suggested, in which, $\frac{v_a^2}{2g}$ is the mean velocity head of approach above the

crest level, and $\frac{v_b^2}{2g}$, the mean velocity head of approach below the crest level. If the velocity conditions are unknown it is necessary to make measurements to find the distribution. This formula is tested by its application to the new experimental data and is found to fit all of them much better than the formulas of Fteley and Stearns, or of Bazin, or the curves of Lyman, or the formula, $Q = 3.33\ L\left(h + \frac{v_a^2}{2g}\right)^{\frac{3}{2}}$. In extreme cases of high heads on low weirs the corrective terms modify as much as 50% the value that would be obtained by the simple Francis formula, $Q = 3.33\ L\ h^{\frac{3}{2}}$, but, nevertheless, they fit the volumetric measurements within about 1 per cent. Other simpler formulas of limited applicability are suggested also. In particular, attention is called to Rehbock's formula, $Q = \frac{2}{3}\ \omega\ L\ h\sqrt{2\,g\,h}$, in which, $\omega = 0.605 + \frac{1}{320\ h - 3} + 0.08\ \frac{h}{d_0}$, in foot-second units ($d_0$ is the height of the weir).

(The experimental data of Francis, Fteley and Stearns, Bazin, etc., cannot be used to test the new formula because no adequate velocity measurements were made).

(12) The use of such a weir properly built, of good head measurements and good auxiliary velocity measurements in the channel of approach, should insure discharge values within 2% of the truth, and in most cases within 1% (see Fig. 18). However, this seems to be quite untrue for many weirs relied on for such a precision by the use of formulas and methods in common use.

(13) The paper in no way presumes to champion the sharp-crested weir as a measuring device superior to others.

Experimental Equipment

The Experimental Canal.—The weirs for the experimental series, A to P, inclusive, (Fig. 1) were in the Hydraulic Laboratory of Cornell University. Inside a concrete canal, 6 ft. wide, was formed a new inner wooden canal approximately 4 ft. 3 in. wide, with flared sides up stream forming an easy approach for the water; and with down-stream sides projected beyond the weir to prevent lateral spreading of the overfalling water. Aeration holes just below, and close to, the crest on the down-stream side served to admit air under the overflowing sheet of water for the low heads. For the higher heads the general construction permitted free access of air under the sheet. Other essential details of the canal are given in Fig. 1.

Water Supply.—The water supply came by gravity from Beebe Lake, a pond of 20 acres formed in Fall Creek by a concrete overfall dam, 26 ft. high, with a spillway 130 ft. wide. During the experiments the water was usually flowing over the spillway, thus insuring nearly a constant head. The watershed was 125 sq. miles. From the base of the dam the water flowed through 192 ft. of 48-in. pipe and then 52 ft. of 30-in. pipe, terminating at the side of the small canal near the up-stream end with its invert even with the bottom of the canal (Fig. 1). For regulating the flow there was a 30-in. hand-

operated gate-valve. The static head on the center of the valve with water at the crest of the dam was 17.5 ft. With water in the small canal barely flowing over the experimental weir, the back pressure in the canal reduced the difference between the head up stream and down stream from the 30-in. valve to an average of about 10 ft.

Baffles.—Instead of allowing the discharge from the valve and the 30-in. pipe to churn around unrestrictedly in the canal, and depending on the usual type of baffle-racks to quiet the flow, a deflecting hood was constructed, to serve both as an enlarging elbow and as an energy killing chamber for the first and most violent part of the commotion (Fig. 1). The water flowing out under the hood was further quieted and directed, first upward, then forward

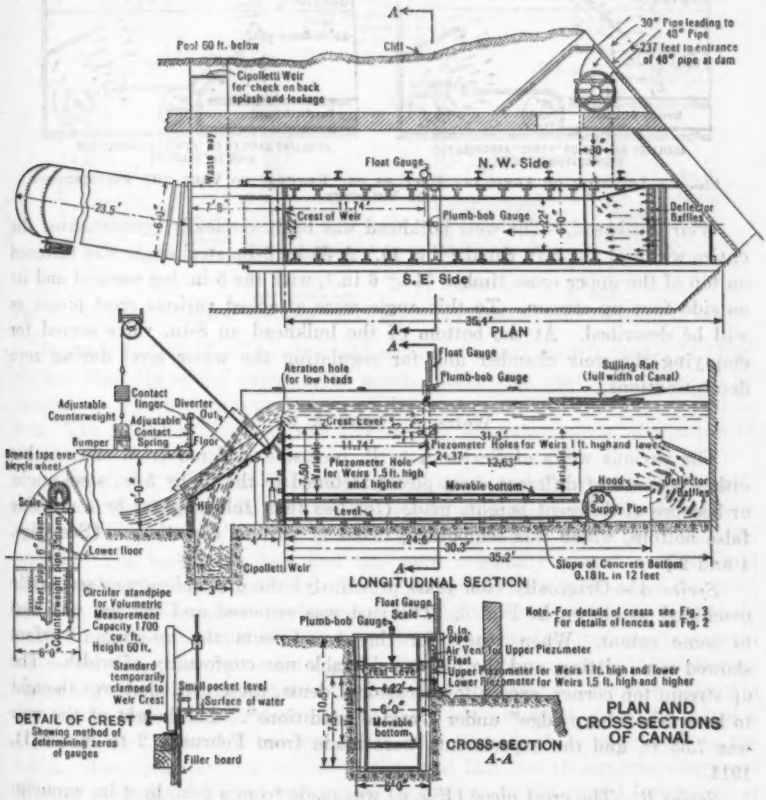

FIG. 1.—GENERAL PLAN OF CANAL, WEIR, AND APPARATUS.

over the top of the hood, by a system of open deflector baffles arranged so as to enlarge the cross-section progressively. This scheme proved remarkably effective except for some tendency to form surface waves at the very highest flows (heads above 2.0 ft. on the weir). A stilling-raft of 2-in. planks floating on top of the water above the deflectors tended to eliminate these waves.

"Fences."—In order to vary the distribution of velocities from the conditions during the so-called "regular" runs with only the permanent baffles controlling the flow, additional controls were built across the canal for some of the higher heads. These were usually in the form of inclined fences (Fig. 2) with 1 by 6-in. horizontal boards spaced by trial at such distances as to secure the desired deflection of a part of the water toward the surface or toward the bottom, as the case might be, always allowing some of it to flow through the open spaces.

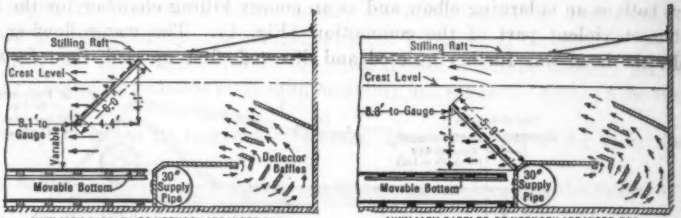

FIG. 2.—LOCATION OF AUXILIARY BAFFLES OR "FENCES" TO VARY THE DISTRIBUTION OF VELOCITY.

Weir Bulkhead.—The weir bulkhead was built vertically square across the entire width of the 6-ft. canal (Fig. 1). A 3½ by 5-in. steel angle was fastened on top of the upper cross timber (4 by 6 in.), with the 5-in. leg vertical and its outside face up stream. To this angle were attached various crest pieces as will be described. At the bottom of the bulkhead an 8-in. valve served for emptying the weir chamber and for regulating the water level during zero determinations.

Weirs of Series A to P

The various weirs of Series A to P, inclusive, differed from one another either because of different crest pieces fastened to the 3½ by 5-in. steel angle, or because of different heights made (for less than full heights) by a movable false bottom, which was temporarily fixed at various elevations. (See Figs. 1 and 2.)

Series A.—Originally (ten years previously) the crest piece was accurately machined, as shown in Fig. 3. The rust was removed and the top was filed to some extent. When ready for the experiments the up-stream surface showed some pitting, and the top, considerable non-conformity of width. The up-stream top corner, except for occasional dents, presented what was thought to be a fair "sharp edge" under "service conditions". The height of the weir was 7.53 ft. and the experiments were made from February 2 to March 11, 1911.

Series B.—The crest piece (Fig. 3) was made from a ⅜-in. by 4-in. wrought-iron plate, planed straight, square, and sharp; and was bolted to the up-stream face of the weir plate for Series A. The height of the weir was 5.51 ft. and the experiments were made from March 25 to 31, 1911.

Dawson, Series I.—The crest piece (Figs. 3 and 4) was the same as that used in Series B, but after two years' service. The up-stream face and top appeared to have suffered considerable corrosion, although the up-stream top

corner showed unevenness rather than rounding. It was more decidedly pitted than the crest piece of Series C. The height of the weir was 5.51 ft. and the experiments were made in January, 1913.

Dawson, Series II.—A $\frac{3}{8}$-in. by 4-in. wrought-iron plate was accurately planed on top and along the up-stream face. The up-stream top corner was square and sharp; the top was flat, $\frac{1}{8}$ in. wide, with its down-stream edge rounded to a $\frac{3}{8}$-in. radius. (See Fig. 3.) The experiments were made while the crest was sharp, smooth, and free from rust. The height of the weir was 5.49 ft. and the experiments were made in May, 1913.

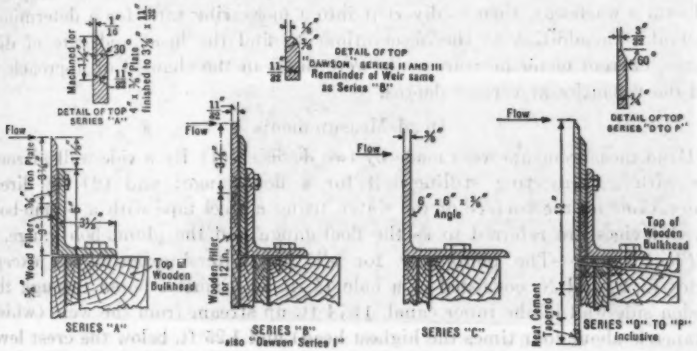

FIG. 3.—DETAILS OF CRESTS, SERIES A, B, C, AND D TO P, ALSO DAWSON SERIES, I, II, AND III.

Dawson, Series III.—The plate and setting was that used in Series II (Fig. 3), but after six weeks' exposure to water. About ten rust nodules could be detected on the up-stream top corner of the crest and slight roughness could be felt. The height of the weir was 5.49 ft. and the experiments were made in June, 1913.

Series C.—A 6 by 6-in. by $\frac{3}{8}$-in. angle (Figs. 3 and 5), a part of the crest piece from the lower 16-ft. weir in the outside experimental canal, served as a crest. This angle was used in place of the 5 by $3\frac{1}{2}$-in. angle previously described. The up-stream face and top were originally planed straight and sharp with a flat top $\frac{3}{8}$ in. wide. The experiments were made after the crest had been in service about twelve years, during which time it was frequently scrubbed and occasionally filed to keep the up-stream face and the up-stream top corner free from rust. During the Series C measurements, the crest part of the angle (see Fig. 5) was in a good state of preservation, showing extensive rusting on the up-stream face near the top, but not what would be called a pitted surface. The top had somewhat of a glaze, making it smooth to the touch. The up-stream top corner of the crest had lost its extreme sharpness so that, while for the most part square and sharp to the casual inspection, it was actually dented and felt dulled and rounded. The height of the weir was 5.34 ft. and the experiments were made from February 8 to April 12, 1914.

Series D to P.—The crest piece was of rolled brass, 12 in. wide and $\frac{1}{4}$ in. thick, and was added to the up-stream face of the crest piece, Series A. The up-stream top corner was straight, square, and sharp. The flat top was care-

fully machined to a $\frac{3}{32}$-in. width and beveled down stream 60° to the horizontal. During the experiments a true sharp corner was carefully maintained by occasional use of a fine carborundum stone. The up-stream face was kept smooth and free from organic deposits. The small shoulder at the lower edge of the plate was filled in with neat cement for a 4-in. depth and tapered to a thin edge. (See Fig. 3.)

Method of Experimentation

General Method.—The procedure was to establish a steady flow of water over the weir at approximately the desired head, permitting the discharge to fall into a wasteway, then to divert it into a measuring tank for a determined interval. In addition to the observations to find the head and rate of discharge, current-meter measurements were made in the channel of approach to find the velocities at various depths.

Head Measurements

Head measurements were made by two devices: (1) By a side-wall piezometer with a connecting stilling-well for a float-gauge; and (2) by direct observations on the surface of the water, using a steel tape with a plumb-bob. These devices are referred to as the float-gauge and the plumb-bob gauge.

Float-Gauge.—The piezometer for all the writers' experiments, except Series E, F, and N, consisted of a hole $1\frac{1}{8}$ in. in diameter bored through the wooden side-wall of the inner canal, 11.74 ft. up stream from the weir (which distance is about four times the highest head) and 1.25 ft. below the crest level (Fig. 1). For Series E, F, and N the piezometer consisted of two such holes (Fig. 1), 9 in. apart horizontally, located 0.25 ft. below the crest level, the mid-point between the holes being vertically above the single hole just described. This different piezometric arrangement was made necessary by the fact that the floor of the channel for Series E, F, and N was above the single piezometer hole. It was deemed desirable to use the same float-gauge for all series, and there was an advantage in construction in having two symmetrically placed holes, with a supposed greater accuracy of readings for these shallow channels with relatively high velocities of approach.

Into the single hole was screwed, horizontally from the outside, a 20-in. length of 1-in. standard wrought-iron pipe, but not projecting through on the inside. At its outer end an up-turned elbow connected it with a vertical, 6-in. wrought-iron pipe, 5 ft. high, serving as a stilling-well for the float-gauge proper (Section *A-A*, Fig. 1).

In the well was a cylindrical copper float, top and bottom ends dished outward so as to shed the water and not to entrap air. The float stem consisted of a $\frac{1}{4}$-in., round, brass rod passing axially entirely through the float, thus materially stiffening it.

The stem was long enough to extend above the top of the 6-in. pipe when the water level in the canal was several inches below the weir-crest level. At the top of this stem a combined index and guide (Fig. 6), traveled along the edge of a brass scale graduated to hundredths of a foot.

For Series A, B, and for 18 runs of Series C, the float was a cylinder, $5\frac{5}{8}$ in. in diameter. For all later work, a 5-in. float was used with four ver-

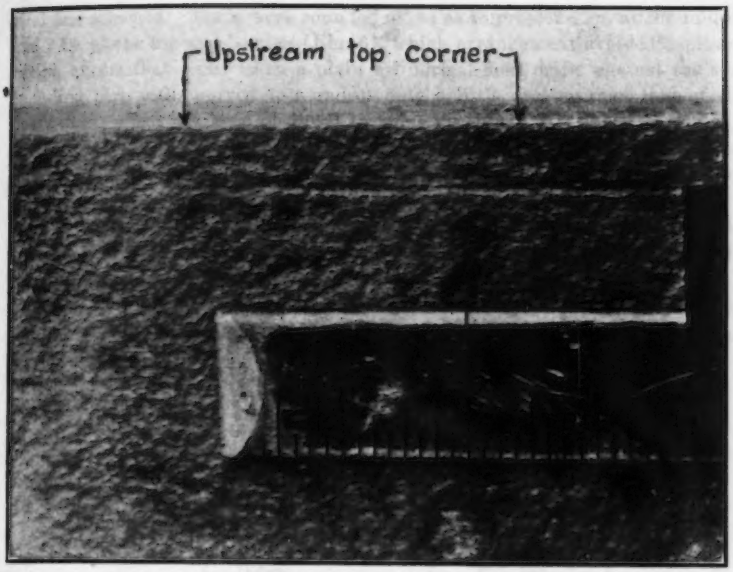

FIG. 4.—CREST PIECE, DAWSON SERIES I (SCHODER AND TURNER SERIES B AFTER TWO YEARS).

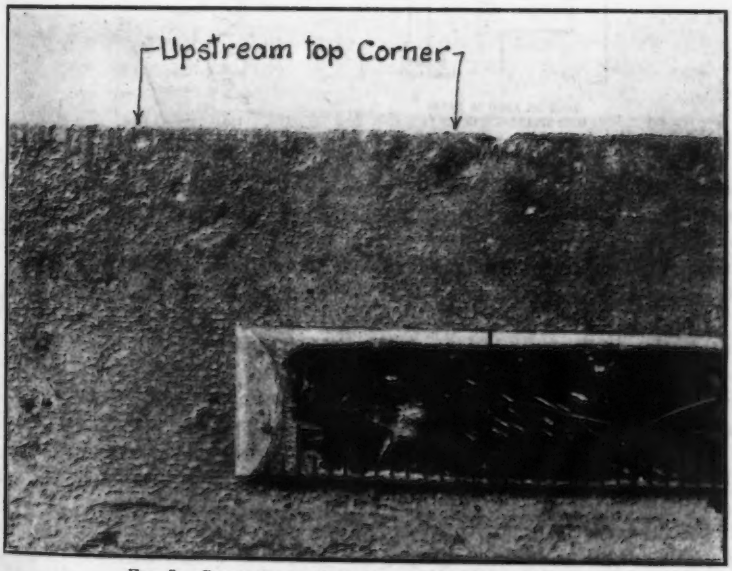

FIG. 5.—CREST PIECE, SCHODER AND TURNER SERIES C.

Upstream Top Corner

Upstream Top Corner

tical fins attached. These were rounded off so as to project ⅜ in. at the middle and ¼ in. at the top and bottom (Fig. 6), which arrangement avoided capillary lifting effects that occur when a plain cylindrical float drifts against the surrounding pipe walls curved to a radius only a little greater than that of the float. This effect has been observed to be as much as 0.003 ft.

For Series E, F, and N, a pair of horizontal 1-in. pipes led from the two piezometer holes to tees in vertical 1-in. pipes, the downward extensions of which on either side led through an elbow and a short horizontal piece to a cross in the 1-in. pipe leading out of the lower piezometer hole. The upward extensions of the vertical pipes reached above the highest water level in the canal and served as air vents. For the lower piezometer the float well itself served as an air vent. Plug cocks in the three 1-in. pipe lines enabled either the lower single-hole piezometer or the upper double-hole one to be connected to the float well.

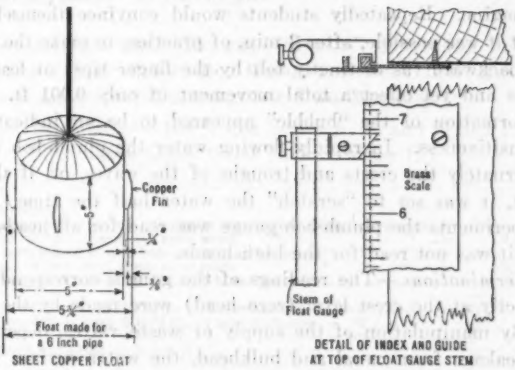

FIG. 6.—FLOAT-GAUGE DETAILS.

In use, the float-gauge has the great merit of giving continuous indications without the necessity of making settings. The degree of damping of the magnitude of waves or surges may be varied by changing the ratio of area of piezometer hole to that of the float well. To remove any mechanical friction or "sticking" of the index the observer at short intervals rapped the framework of the scale.

With high heads, or rapid flows in the channel of approach, the alternating high and low readings of the float-gauge were taken. Usually, the time interval between successive high and low indications was 15 to 40 sec. As a general rule, several readings of the float-gauge were taken just before, during, and after the discharge measurement.

*Plumb-Bob Gauge.**—This gauge (Fig. 1) consisted of a steel tape, graduated to hundredths of a foot, to the end of which was attached a brass plumb-bob. A 4 by 12-in. plank, securely bolted, across the 6-ft. canal carried the "tape block", having on its top a brass plate grooved to admit the steel tape

* The plumb-bob gauge used was substantially as designed by Gardner S. Williams, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXVII (1914), p. 1261.

and marked with an index line. The tape hung over the rounded end of the block the same distance up stream from the weir as the piezometer holes of the float-gauge. In use, the plumb-bob was rapidly lowered by unreeling the tape which was then slipped into the groove where it was lightly held with the finger tips or thumb of the left hand. The right hand steadied by the back of the tape block then fed the tape down or up, as desired by manipulation between thumb and forefinger. The bob was slowly raised out of the water until the "bubble" at the point "broke" (that is, disappeared), the point then being about 0.005 ft. above the water surface. The tape was then very slowly fed out until the "bubble" formed at the point of the bob indicating contact with the surface, for which position the reading was taken. This phenomenon was of pronounced visibility, even when the observer was 10 or 15 ft. above the water surface.

The device proved to be far more sensitive than one unacquainted with it would imagine. Repeatedly students would convince themselves to their surprise that it was possible, after 3 min. of practice, to make the tape "jump" forward or backward (as distinctly felt by the finger tips) at least three successive times and yet effect a total movement of only 0.001 ft. With quiet water the formation of the "bubble" appeared to be an indicator of corresponding sensitiveness. In rapidly flowing water the plumb-bob was adjusted to show alternately the crests and troughs of the waves, or, if these were of small height, it was set to "scratch" the water half the time. For several series of experiments the plumb-bob gauge was read for all heads, but, in the later series, it was not read for the high heads.

Zero Determinations.—The readings of the gauges corresponding to water surfaces exactly at the crest level (zero head) were made by the "water-level method". By manipulation of the supply or waste valves, depending on the amount of leakage from canal and bulkhead, the water surface in the canal was held practically steady just below the level of the crest or perhaps a few thousandths of a foot above the crest.

A hook-gauge, temporarily clamped to the crest (Fig. 1), served to measure the exact difference of elevation between water surface and the crest. The point of the hook was adjusted so as to be about 1½ in. up stream from the crest. A small pocket level with a sensitive bubble was nearly balanced across the crest piece with the up-stream end resting lightly on the point of the hook. The hook-gauge was then adjusted so as to center the bubble, and the hook-gauge reading was taken. The procedure was repeated with the spirit-level reversed. Then, on signal from the hook-gauge observer, when he made contact with the water surface, the float-gauge was read. This was repeated six times and then the crest-gauge was again read for crest level.

The whole procedure was repeated for the plumb-bob gauge (there being but two observers). Such a determination was made with the hook-gauge at the middle of the crest and also at the one-sixth points out from either canal wall, so that there was an independent zero determination for each one-third of the crest length for each gauge. For some series several such complete determinations were made.

The benefits of these careful zero determinations appear in the close agreement of heads as determined by the two types of gauges. (See summary in Table 2.)

Discharge Measurements

The discharge measurements of the sheet of water flowing over the weir were obtained by diverting it for an appropriate interval of time from the wasteway into the calibrated tank.

"Diverter."—The diverter (Fig. 1) was an inclined rectangular wooden apron, 8. ft. long, fitted with sheet-iron sides, 15 in. high, so as to form a steep chute. It was hinged near its lower edge to the top of a bulkhead built at the entrance to the standpipe.

Hoisting Tackle.—This consisted of a 1-in. rope passing over a pulley, the up-stream end being attached to the apron by rod-links and the other end sustaining an adjustable counterweight.

TABLE 2.—CORRESPONDENCE BETWEEN "HEADS" MEASURED (1) BY FLOAT-GAUGE; AND (2) BY PLUMB-BOB GAUGE FOR VERY LOW HEADS, SHOWING THE CLOSE AGREEMENT BETWEEN THE TWO METHODS.

(Heads below 0.100 ft. for Series A, B, and D to P; heads below 0.200 ft. for Series C.)

Series D to P, Inclusive:		
Number of runs where plumb-bob gauge was higher than float-gauge =	70	
Average head, in feet, that these runs were " " " " =		0.0004
Number of runs where plumb-bob gauge was lower " " " " =	39	
Average head, in feet, that these runs were " " " " =		0.0004
Number of runs with the same head by both gauges =	5	
Total number of runs considered in this comparison =	114	
Number of runs with 0.0005 ft., or less variation = 92		
" " " " 0.0010 ft., " " " = 108		
Maximum variation = 0.0016 ft.		
Series A, B, C:		
1st.—Using old type float (without fins):		
Number of runs where plumb-bob gauge was higher than float-gauge =	9	
Average head, in feet, that these runs were " " " " =		0.0011
Number of runs where plumb-bob gauge was lower " " " " =	16	
Average head, in feet, that these runs were " " " " =		0.0008
Number of runs with the same head by both gauges =	1	
Total number of runs considered in this comparison =	26	
Number of runs with 0.0005 ft., or less variation = 11		
" " " " 0.0010 ft. " " " = 19		
Maximum variation = 0.0022 ft.		
Series C:		
2d.—Using new type float (with fins):		
Number of runs where plumb-bob gauge was higher than float-gauge =	14	
Average head, in feet, that these runs were " " " " =		0.0005
Number of runs where plumb-bob gauge was lower " " " " =	15	
Average head, in feet, that these runs were " " " " =		0.0002
Number of runs with the same head by both gauges =	2	
Total number of runs in this comparison =	31	
Number of runs with 0.0005 ft., or less variation = 24		
" " " " 0.0010 ft., " " " = 29		
Maximum variation 0.0011 ft.		

A 1-ton spur-geared chain block served to hoist the counterweight and to sustain it until after the diverter was dropped from the "up" to the "down" position. It also served as a control while adjusting the diverter contact.

To hold the diverter in the "up" position when the counterweight was hoisted, the bent end of an iron rod was "hooked" over the lip of the diverter,

the other end being hooked over an iron railing. This holding rod was called the "trigger". A small rope, tied to the trigger near the lower end, passed over the top of the extension side of the inner canal where it sustained a small counterweight. A pull on the diverter rope by one of the experimenters, bracing himself against the railing, raised the diverter slightly so that the trigger could be pulled out of the way by the small counterweight. The diverter could then be "dropped" through the sheet of water, and the large counterweight released by the chain block until it hung free. This counterweight had been adjusted previously so that the force of the sheet of water would hold the diverter down, but a slight downward pull on the counterweight would cause the diverter to rise through the sheet of water.

The diverter chute was 5.5 ft. wide, being narrow enough to have some clearance inside the 6-ft. canal and inside the entrance to the 6-ft. standpipe, and yet wide enough to project about 6 in. outside the over-head extensions of the side-walls of the inner canal. When the diverter was in the "down" position, it was supported by a "bumper" consisting of a 4 by 4-in. cross-piece resting on side posts independent of the weir bulkhead. The upper edge or lip of the diverter was about 1 in. below the down-stream edge of the small fixed sloping apron on the weir bulkhead. This arrangement caught all the water, even at low heads, except those between about 0.60 and 0.75 ft., which had to be avoided because the sheet of water struck the diverter so close to the upper edge that considerable water splashed backward and upward through the clearance space between the diverter and the bulkhead apron.

Wasteway.—The wasteway (Fig. 1) acted as an intercepter for any leakage from the weir bulkhead, so that the measuring tank was unaffected by such leakage. With the highest heads, in Series A, B, C, and Dawson's Series I, II and III (more than 2 ft. over the weir), when the diverter was in the "up" position with water flowing to waste, there was considerable turmoil in the wasteway, and a small quantity of water found its way into the stand-pipe through the hinge and over the bulkhead through the clearance spaces outside the diverter. This small leakage was accounted for in the regular course of the observations for initial and final water levels in the stand-pipe measuring tank. When the diverter was in the "down" position (water flowing into the stand-pipe), there was, at the highest heads of Series A, B, and C, a small back-splash through the clearance spaces out from the standpipe into the wasteway. To account for this, the law of total leakage into the wasteway was found by observing the heads on the 12-in., Cipolletti weir notch (Fig. 1) in a bulkhead across the outer end of the wasteway channel, for various heights of water back of the experimental weir, ranging from several feet below the weir crest to the level of the highest heads used in the experiments.

For water levels above the crest of the weir the diverter was in the "down" position, as during the discharge measurements. Thus, the law of leakage variation through the bulkhead for different heads was clearly established, and any excess was charged to back-splash from the stand-pipe. As a check, actual observations of conditions were made by a man going into the wasteway under the diverter while the flow was proceeding. Both the weir bulkhead and the

bulkhead at the stand-pipe entry were frequently examined, and conditions always found to be as assumed for the correction measurements and calculations.

This back-splash from the stand-pipe into the wasteway at high heads was practically eliminated for the writers' Series D to O, by sealing all clearance cracks of appreciable size with thin wooden strips.

Stand-Pipe Measuring Tank.—This tank (Fig. 1) was a circular steel stand-pipe, 6 ft. in diameter, 60 ft. high, made of cylindrical sections, 5 ft. high. It was calibrated in two ways: (1) By measuring its outside girth by a steel tape with proper allowance for thickness of metal, vertical position of the lap joints, volumes of recesses inside at various attached saddle nozzles with their gate-valves, rivet heads, etc.; and (2) by repeatedly filling with water and then draining off successive quantities into a precisely calibrated tank with a capacity of 400 cu. ft. The initial level and the intermediate levels were varied so that the final calibration points covered the entire 60-ft. vertical range by 1-ft. intervals. These two calibrations were referred to tape and scale readings on the stand-pipe float-gauge.

Stand-Pipe Float-Gauge.—The float was of copper, torpedo-shaped, moving in a 6-in. vertical pipe set alongside the stand-pipe and connected to it at the bottom by a 2-in. pipe. A bronze ribbon tape was attached to the top of the float and passed up out of the 6-in. pipe, over a bicycle wheel and down into a 3-in. vertical pipe which served as a guide for the counterweight on the other end of the tape (Fig. 1). On top of the 6-in. pipe, alongside the vertical tape, was mounted an auxiliary brass scale divided into 0.1 and 0.01 ft. The bronze tape had graduations at 1-ft. intervals. This arrangement permitted readings by estimation to 0.001 ft., which extent of change of level in the stand-pipe and float pipe was readily indicated by a corresponding movement of the tape. From time to time the indications of the bronze tape were checked, in operating position, by comparing the indicated difference of elevation between near-full and near-empty conditions with the difference by steel-tape measurements between water gauges near the top and bottom.

The adopted calibration was that by the water-filling method with direct readings on the float-gauge, just as in the experiments proper. This calibration was based on precise weighings of a succession of 5-cu. ft. quantities into the lower 400-cu. ft. rectangular tank for its calibration. This lower tank was also calibrated by dimensional measurements. The two methods agreed within 0.1 per cent.

The precise determination of the inside dimensions of the stand-pipe involved great difficulties, and yet the two calibrations agreed within 0.2 per cent.

Time Measurements.—The durations of the diversions into the stand-pipe were measured, except for low heads (for which an ordinary watch was used) by a ribbon electric chronograph, one pen actuated by a seconds pendulum and the other by a diverter contact.

This contact device (Fig. 1) consisted of a weight, free to move vertically on a pair of fixed iron guide-rods, and suspended by a wire passing over a

small grooved pulley and fastened to the diverter. The weights carried a brass contact finger moving between two leaves of spring brass mounted on a metal block that could be clamped anywhere along a second pair of vertical guide-rods. The position of the gap was adjusted (for each different head on the weir) so that the contact point closed the circuit just as the lip of the diverter passed through the middle of the sheet of water. In some of the first experiments two contacts were used, one adjusted for the diverter lip at the lower, and the other, at the upper, surface of the sheet of water.

With an effective volume of about 1 200 cu. ft. available in the stand-pipe, a measurement at the highest head (60 cu. ft. per sec.) covered 20 sec. With a head of 1 ft., the duration was about 80 sec. Usually, for the highest heads, the discharge was measured five times in succession. The chronograph was used for measurements up to 2 min. in duration. Low-head measurements lasting 3 min. or more were timed by a well-regulated watch, the second hand of which made 5 visible jumps per sec. To avoid effects of eccentricity of the seconds' dial, the runs were made to cover an exact number of full minutes.

Procedure in a Typical Run

As a rule, neither runs nor zero determinations were made until after the weir bulkhead and canal walls had become thoroughly water-soaked. Customarily, water was flowing over the weir at all times, whether or not experiments were in progress.

At the start of a run the 30-in. valve was opened so as to give the desired head as shown by the weir float-gauge. Time was allowed for the head to become steady; this developed almost as fast as the valve could be opened for the high heads, but required as long as 10 or 15 min. for the extremely low heads. The leakage through the weir bulkhead into the wasteway materially hastened the adjustment for these low heads.

The diverter contact was then adjusted to a point midway between temporary chalk marks made on the guide-rods as the diverter was slowly moved through the sheet of water by the chain-block control. Also, the counterweight was regulated to suit the volume of water flowing.

The stand-pipe, having previously been partly filled with water, was now nearly emptied by throwing into the "open" position the lever of a 4-way valve which controlled the 36-in. hydraulic valve at the bottom of the stand-pipe (60 ft. below the operating floor). While the stand-pipe was emptying, the control lever was thrown into the "close" position, the operation being so timed that the 36-in. valve would be completely closed with at least 5 ft. of water remaining in the stand-pipe. This process of closing "under pressure" materially reduced the leakage from the closed valve.

Usually three or more readings at ½-min. intervals were taken on the stand-pipe gauge for the initial water level; also the weir float and plumb-bob gauges were read several times. The chronograph was then started, the diverter dropped, and the counterweight released to hang free. (Usually one observer read the weir float-gauge while the flow was going into the stand-pipe. For low heads both the float-gauge and the plumb-bob tape were read several times

alternately. For the high heads the duration of the volumetric measurement was too short to permit readings of the plumb-bob gauge.) At the end of the desired interval of time the counterweight was pulled, moving the diverter up through the sheet of water, and the trigger was hooked over the lip of the diverter. The chronograph was then stopped and the ribbon examined to make sure the record had no flaws.

Just 1 min. after the diverter went up, allowing this time for settlement in the stand-pipe and its float-pipe, readings were taken for final water level, usually three or more at ½-min. intervals. Immediately after the diverter went up, readings of the weir head-gauges were taken, as at the beginning. The stand-pipe was then emptied to about the 5-ft. level, ready for another run. Some typical original laboratory notes and primary calculations are given in Table 3.

Primary Calculations from Observed Data

Leakage Corrections.—It was the practice to make corrections systematically even when they seem negligibly small, as in most cases. From Column (3) of Table 3, it is seen that in Run 55, Series D, the surface of the water in the stand-pipe measuring tank was rising at the beginning (before the diversion into the tank) at the rate of 0.032 ft. per min., and at the end (after the run) at the rate of 0.025 ft. per min., both values being recorded in Column (8). This rise was due to some leakage into the stand-pipe at the diverter hinge from splash in the wasteway.

It was known from many observations that there was a negligible leakage from the valve at the bottom of the stand-pipe with low heights of water, increasing to a small but appreciable quantity for the filled stand-pipe. Consequently, as the data show, at the end the diminution in the rate of rise represents the rate of fall (relatively to the beginning) due to leakage from the stand-pipe—in this case, 0.007 ft. per min. With no leakage at the start and 0.007 ft. per min. at the end, an average of 0.0035 ft. per min. is used for the $\frac{39}{60}$ min. of the volumetric measurement. Then to the last of the initial readings (here taken 1 min. before the diverter was dropped) is added 0.032 ft. to give the initial reading at the instant of diversion. To the first of the final readings (here taken 2 min. after the diverter was raised) is added the 0.002 ft. to offset leakage from the stand-pipe during the run; finally, $2 \times 0.025 = 0.050$ ft. is subtracted on account of the leakage into the stand-pipe during the 2 min. elapsed. In this example, the corrections are small, but with heads of more than 2 ft. they were considerable.

The values in Column (9) of Table 3 are taken from the calibration curve of the stand-pipe for the corrected initial and final water levels. This curve permitted readings of heights to 0.005 ft. and of volumes to 0.1 cu. ft. In Column (4) the times of day for diverter "in" and "out" are noted as read on an ordinary watch to the nearest second. The exact duration as scaled from the chronograph ribbon is also recorded. In Column (5) the readings of the weir float-gauge before, during, and after the run are shown. Column (6) gives the readings of the plumb-bob gauge before and after the run, and shows that no readings by this gauge were taken during Run D-55.

TABLE 3.—SAMPLES OF ORIGINAL NOTES, SHOWING OBSERVED DATA AND PRIMARY CALCULATIONS.

SERIES D WEIR, 7.5 FEET HIGH.

OBSERVED DATA.						COMPUTED DATA.			
Number of run.	Time of day.	Stand-pipe float-gauge, in feet.	Duration of volumetric measurement.	Weir float-gauge, in feet.	Weir plumb-bob gauge, in feet.	Weir gauge averages and head calculations, in feet.	Stand-pipe rates of fall or rise and adjusted initial and final readings.	Contents, in cubic feet, initial, final, and difference.	Discharge, Q, in cubic feet per second.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
55	10 : 31 : 00	4.633		2.511	27.136		Beginning		
	10 : 31 : 15	4.642		2.490	27.136	Float :	+ 0.032 ft. per min.	.....	.....
	10 : 31 : 30	4.651		2.506	27.130	2.4999	Ending		
	10 : 31 : 45	4.657		2.496	27.142	0.6949	+ 0.025 ft. per min.	.....	.....
	10 : 32 : 00	4.665		2.509	.....	1.8050	2 | 0.007		
	.....	.....	In, 33 : 00	2.493	.....		0.0035	.....	.....
	.....	.....	Out, 33 : 40	2.500	.....	Plumb-bob :	$0.0035 \times \frac{39}{60} = 0.002$	.....	.....
	10 : 35 : 40	52.478	Chronograph shows 38.99 sec.	2.488	.....	27.1342	4.665	.....	.....
	10 : 35 : 55	52.485		2.498	27.135	25.3249	+ 0.032	.....	.....
	10 : 36 : 10	52.492		2.506	27.133	1.8048	4.697 initial	125.6	.....
	10 : 36 : 25	52.498		2.497	27.129		52.478	.....	31.9864
	10 : 36 : 40	52.503		2.505	27.133		− 0.050	.....	.....
	.....	.....		.....	.....		+ 0.002	.....	.....
	.....	.....		.....	.....		52.430 final	1489.7	.....
	.....	.....		.....	.....			1364.1	.....
	.....	.....		.....	.....			Difference	

TABLE 3.—(*Continued.*)

SERIES J WEIR, 4.0 FEET HIGH.

OBSERVED DATA.						COMPUTED DATA.			
Number of run.	Time of day.	Stand-pipe float-gauge, in feet.	Duration of volumetric measurement.	Weir float-gauge, in feet.	Weir plumb-bob gauge, in feet.	Weir gauge averages and head calculations, in feet.	Stand-pipe rates of fall or rise and adjusted initial and final readings.	Contents, in cubic feet, initial, final, and difference.	Discharge, Q, in cubic feet per second.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
14	1:07:00	6.293		0.875	25.508	Float:	Beginning	.....	.....
	1:31:00	6.280		0.8745	25.508	0.8747	— 0.0005 ft. per min.	.....	.....
	.....	.....	In, 1:31:00	0.875	25.5075	0.6959	Ending	.....	.....
	.....	.....	Out, 1:41:00	0.8745	25.5075		— 0.0055 ft. per min.	.....	.....
	1:48:00	28.754	600 sec.	0.8745	25.508	0.1788		.....	.....
	1:45:00	28.743		0.875	25.508	Plumb-bob:	2|—0.006	.....	.....
	.....	.....		0.8745	25.508	25.5078	— 0.003	.....	.....
	.....	.....		0.8745	25.5075	25.8290	6.280 initial	170.4	.....
	.....	.....		.....	.....		28.754	.....	.....
	.....	.....		.....	.....	0.1788	+ 0.011	.....	1.0703
	.....	.....		.....	.....		+ 0.030	812.6	.....
	.....	.....		.....	.....		28.795 final	642.2	.....
	.....	.....		.....	.....			Difference	.....

Column (7) shows the average gauge readings, and the subtraction of the zero readings to obtain the heads on the weir crest. In Series D it was decided to average all the weir float-gauge readings rather than to use only those taken during the run, although this latter procedure was the usual one when sufficient readings were available. In this case there were five consecutive runs, Nos. 51 to 55, inclusive, at the same valve setting. In only three runs, Nos. 51, 52, and 55, was the float-gauge read during the run, but in all of them readings were taken just before and just after the run, as was the case also for the plumb-bob gauge. The readings during Runs Nos. 51, 52, and 55 by float-gauge as compared to the average of readings before, during, and after the test, showed a deficiency of 0.0009 ft., or 1 part in 2 000; as compared to the grand average by both gauges, the deficiency was only 0.0004 ft. The average head by float-gauge in Runs Nos. 53 and 54 (readings taken only before and after the run) shows exact agreement (h = 1.8039 ft.) with the grand average.

In Run No. 14, Table 3, covering Series J (typical low head), all the weir head readings were taken during the volumetric measurement, which lasted 10 min. The duration was controlled by an ordinary well-regulated watch, the observer moving the diverter to the nearest 0.2 sec. The steadiness of the water surface in the channel of approach tempted the observer to estimate 0.0005 ft. in reading the gauges.

For the very highest heads the leakage into the stand-pipe from the waste-way before and after the measurement was greater than in Run No. D-55. For heads of 2.0, 2.25, 2.50, and 2.75 ft. in Series D the rates of rise in the stand-pipe were, respectively, 0.075, 0.45, 1.5, and 2.5 ft. per min., so that great care was used in noting the exact second the diverter went in and out. Also, at least five readings on the stand-pipe gauge were taken at exact 15-sec. intervals before and after the measurement, in order to determine the rate of rise closely. The weir-head gauges were each read, usually six times, immediately before and, again, immediately after the measurement.

Velocity Measurements

Velocity measurements in the channel of approach were made for the relatively higher heads, just before or just after the volumetric measurements, using a small Price electric current meter with a telephone receiver. The meter was held in position by an iron rod graduated to feet and tenths, and was operated through the same cross-section in which the head was measured. The meter was rated several times during the course of the experiments and found to remain substantially constant in its indications.

Velocities.—For most of the writers' Series A to P, only the velocities along a single vertical element at mid-stream were observed. For Series F, velocities 0.5 ft. from either side-wall were read for a head of 2.36 ft. on the weir. For Series K, velocities in five verticals were observed at 2-ft. head. Fig. 42 shows the horizontal distribution of velocities. Appendix B tabulates the results of all velocity measurements, and Figs. 33 to 47 show the curves of velocity distribution.

For Series N (0.75-ft. weir) and Series K (5.5-ft. weir) additional vertical mid-stream traverses were made 6 ft. up stream from the weir to determine whether any considerable re-arrangement of velocities had been effected as the flow approached the weir between the sections, 11.74 ft. and 6 ft. up stream. Figs. 38 and 43 show the comparative distributions. Little change is seen for the 2-ft. head of Series K; but in three of the four cases of Series N there is an indication of a slowing up of the mid-channel velocities, probably at the expense of increasing wall velocities.

With a weir, 3.76 ft. high and 5 ft. long, and with screens 17.7 ft. up stream from the weir, Fteley and Stearns made three velocity measurements for a head of 0.78 ft., in each of three verticals at 16 ft. and also at 8 ft., up stream from the weir. They conclude* that "8 ft. is not a sufficient distance to cause even a partial re-arrangement, such as might be looked for."

Fig. 48 shows a velocity distribution measured by Fteley and Stearns 5 ft. up stream from their 6.55-ft. weir, 19 ft. long, with a head of 1.304 ft.† This measurement stands alone as a definite velocity traverse showing channel conditions in the classic experiments of Francis, Fteley and Stearns, and Bazin.

Adjustment of Velocities.—The average head on the weir during a set of current-meter measurements usually differed slightly from the head during the corresponding volumetric measurements (Appendix B), so that, to obtain the discharge during the current-meter measurements, the observed discharge by volumetric measurement needed to be corrected slightly, according to the law of variation between Q and h for the particular weir at the particular head. (For the high weirs, Q varies nearly as $h^{\frac{3}{2}}$. For the lower weirs at the high heads, the exponent of h is appreciably more than $\frac{3}{2}$, being 1.85 in Series E. It was found for each weir by logarithmic plotting of the values actually measured.)

The true mean velocity of approach prevailing during the current-meter measurements was then calculated by dividing the adjusted volumetric discharge per foot of width by the depth of water in the channel of approach. In general, the mean velocity as found by current meter (above a single vertical element) differed from this true mean velocity by volumetric measurement. The ratio was computed and assumed to apply to each of the component parts of the curve of velocity distribution in the vertical, such as V_a, the mean velocity above the crest level, and V_b, the mean velocity below the crest level.

In a few cases (see Appendix B (Table 55)), the range of velocities was so extreme that the mean of the velocity heads is sensibly in excess of the head corresponding to the mean velocity. In such cases both values are shown, but the mean of the velocity heads is used in subsequent computations.

New Experimental Data by Others

Other experimental data on weirs without end contraction, herein published for the first time, are parts of the results of experiments made at Cornell University by Messrs. Charles L. Walker and Carl R. Weidner (1904)

* *Transactions*, Am. Soc. C. E., Vol. XII (1883), p. 38.
† *Loc. cit.*, p. 74.

while Senior students; and the following men while Graduate Students in Civil Engineering: William F. Martin (1906); Francis M. Dawson (1913); Henry R. J. Meyer and Russell A. See (1914); Jacob O. Jones (1915); and Charles M. Weber (1915).* All these experiments were on sharp crests except the rounded crests of Dawson and Jones.

The writers by personal association had intimate knowledge of the details of all these experiments. The first mentioned experimenters did their work under Gardner S. Williams, M. Am. Soc. C. E., in charge of Experimental Hydraulics at Cornell University. All the others worked with one of the writers in charge.

Walker and Weidner's Experiments.—These experiments showed the effects on the discharge caused by the width (thickness) of the crest piece ($\frac{1}{64}$ in. to 2 in.) at low heads (0.02 to 0.13 ft.). They were made on a weir 0.6 ft. high, set in a wooden flume, 0.895 ft. wide and 12 ft. long. The weir bulkheads for the $\frac{1}{64}$-in. and the $\frac{1}{8}$-in. thicknesses consisted of brass plates, and the $\frac{1}{4}$-in. crest was the edge of a varnished cherry wood plate set against the brass bulkhead with the $\frac{1}{64}$-in. crest. The water supply was by gravity at low velocity through a 6-in. pipe from an auxiliary low-level tank fed by a 2-in. pipe from a tank 45 ft. above, the water level of which was kept nearly constant by a float-valve. Heads were measured by a hook-gauge in the channel of approach 1.58 ft. up stream from the weir. The scale was graduated to 0.0025 ft. and a vernier permitted readings to 0.0001 ft. Discharges were measured by diverting established flows into a weighing tank of 2 000 lb. capacity. Periods of time were measured with an ordinary watch. The data are presented on Figs. 26 and 29, but are not tabulated.

Martin's Experiments.—Martin made his experiments to study the effects on discharge caused by the width of the channel of approach ($1\frac{1}{2}$ in. to 2 ft.). Only the data for the weir in his widest flume, 2 ft., are given here (Appendix A (Table 28) and Fig. 49). The weir was 1.2 ft. high with a double set of baffles, 10 ft. up stream. The crest piece was a brass plate, $\frac{1}{8}$ in. thick, its edge milled to a straight-edge, set flush with the up-stream face of the weir bulkhead. The water supply was by gravity, from a 12-in. connection at the bottom of the 6-ft. stand-pipe already described. At this connection there was a 12-in. gate-valve and an up-turned elbow feeding a 12-in. riser pipe, 12 ft. long, terminating in the bottom of the flume 2.5 ft. from the up-stream end.

Heads were measured 7.25 ft. up stream from the weir by two gauges: (1) A plumb-bob gauge for direct readings of the water surface in the flume; and (2) a piezometer consisting of a 1-in. hole through the side wall near the bottom. Into this hole a $\frac{3}{4}$-in. pipe was screwed from the outside so as to be exactly flush on the inside. The pipe led into the bottom of a cylindrical glass jar, 8 in. in diameter and 2 ft. high, used as a stilling-well, in which the water surface was read by a point-gauge. The given heads are the average for the two gauges, on each of which three to six separate readings were made during the diversion into the tank.

Discharges were measured by diverting the established flows into a lead-lined, 10 by 16-ft. measuring tank, 5 ft. deep, calibrated by withdrawing suc-

* Messrs. Walker, Weidner, Dawson, Meyer, See, Jones, and Weber are now members of the Society.

cessive 5-cu. ft. batches (by weight). Depths of water in the tank were measured by two plumb-bob gauges. All durations were measured both by a good stop-watch and by an ordinary watch read to 0.2 sec.

Dawson's Experiments.—These experiments relating to effects on discharge caused by small roundings of the up-stream top corner of the crest piece, covered sharp and nearly sharp crests and roundings to radii ranging from $\frac{1}{8}$ in. to $1\frac{1}{2}$ in. (Fig. 3). The same flume, apparatus, and methods were used as in the writers' experiments. Only the data for the sharp and nearly sharp crests are given in Appendix A (Tables 25 to 27), being plotted on Figs. 8 and 49, but the data for some of the roundings are represented on Figs. 27, 28, and 29.

Meyer and See's Experiments.—Meyer and See investigated the effects on discharge caused by the height of the weir (covering somewhat the same ground as Bazin's volumetric experiments). These tests were made in a concrete flume* (Fig. 7) 2 ft. wide, 4 ft. deep, and 30 ft. long. There were two sets of experiments. In Set I, the weir bulkhead was progressively lowered, the fixed concrete bottom of the flume being the bottom of the channels of approach. In Set II, the weir bulkhead remained 2.5 ft. high, as in Series A of Set I, but an auxiliary movable wooden bottom (Fig. 7) was introduced and lowered progressively from near crest level to 1.8 ft. below crest level. The water supply was by gravity from Beebe Lake through a 10-in. pipe leading from the 30-in. pipe.

Heads were measured by two float-gauges each connected, in Set I, to one of opposite piezometer holes formed by 1-in. pipes extending through the concrete sides of the flume close to the bottom. The float-pipes and floats were of the type used in the writers' experiments. In Set II, the piezometer holes were in the wooden bottom and a flexible connection was secured by using short lengths of $\frac{3}{4}$-in. rubber hose between $\frac{1}{2}$-in. pipe nipples screwed into the wooden bottom from below and similar nipples cemented into the piezometer holes used in Set I. The piezometer holes of Set II were located 6 in. on either side of the center line of the flume, the same distance from the weir as in Set I.

Discharges were measured by diversion of the established flow into one of two tanks, either a cylindrical tank of galvanized sheet iron of 70 cu. ft. capacity, or a rectangular concrete tank of 400 cu. ft. capacity. The discharge from the weirs fell into a large hopper funnel (Fig. 7) forming the upper end of a down-spout free to swing at its lower end thus turning the water to waste or into one of the tanks.

Durations of time were measured by an electric chronograph substantially as described for the writers' experiments. Velocities in the channel of approach were measured for only a few cases, in Set I, namely, for three heads each in Series C, E, and G. The data are given in Appendix B (Table 55) and plotted on Fig 47.

Jones' Experiments.†—These tests regarding effects on discharge caused by extremely small roundings of the up-stream top corner of the crest piece

* The flume and equipment were briefly described in *Engineering News-Record*, September 2, 1920, p. 452.

† A synopsis of Jones' results appeared in the *Cornell Civil Engineer*, November, 1917.

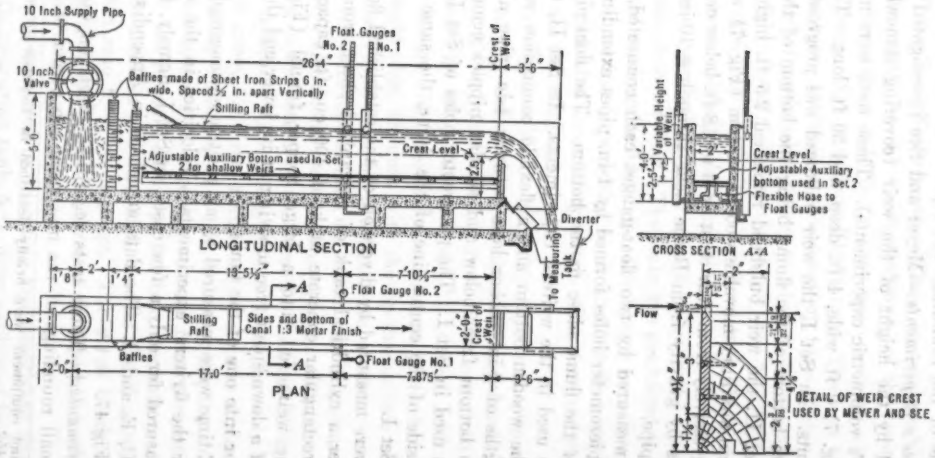

FIG. 7.—PLAN AND SECTIONS, CONCRETE FLUME.

were made with the same flume and measuring devices as used by Meyer and See (Fig. 7), with a weir 2.5 ft. high. The crest piece was a rolled brass plate, ¼ in. by 12 in., and 2 ft. 0 ⅞ in. long—other details as shown on Fig. 7. At first, the up-stream top corner was square and sharp. Then, progressively, the edge was "broken" and approximately rounded more and more for the succeeding series of experiments by light rubbings with a fine emery cloth. To measure the degree of rounding, impressions were taken with dental wax, and a section of the wax was micro-photographed. The data are given in Appendix A (Tables 42 to 49, inclusive) and on Fig. 30. Velocity measurements are given in Appendix B (Table 55), but the data are not plotted.

Weber's Experiments.—The effects on discharge due to the degree of roughness of the up-stream face of the weir near the crest, were made by Weber in conjunction with Jones, using the same height of weir and type of crest piece. The roughness was made by cutting a series of adjoining 60° V-grooves horizontally across the crest plate, starting in Series II, with 96 grooves per in. for 4 in. vertically below the crest, leaving the up-stream top corner square and sharp (Fig. 7). This kept the summits between the grooves in the same plane as for the smooth plate, and avoided a possible criticism of Barr's method of adding shellac and sand to the face.

The next degree of roughness, for Series III, was obtained by cutting 48 grooves per in., removing every other one of the smaller ridges; then, for Series IV, 24 grooves per in. were cut; and, finally, for Series V, as a modification of this last condition, the bottom 8-in. part of the brass plate, hitherto left smooth, was roughened by cutting 15 grooves per in. Series VI reproduced the conditions of Series III, 48 grooves per in. for the upper 4 in. of a 12-in. plate, with the lower 8 in. smooth. Weber's data are given in Appendix A (Tables 50 to 54, inclusive) and on Fig. 32. The velocity measurements are given in Appendix B (Table 55), but are not plotted.

Discussion on the New Data

General

In Appendix A (Tables 9 to 24, inclusive) are given measured heads and discharges per foot of length of weir for the writers' weirs; and also the percentage departures of the measured discharges from those computed for the same head by the simple Francis formula, $Q = 3.33\ L\ h^{\frac{3}{2}}$ (in foot-second units), or $Q = \frac{2}{3} \times 0.623\ L\ h \sqrt{2\ g\ h}$.

The latter part of Appendix A (Tables 25 to 54, inclusive) gives corresponding data for the experiments by Walker and Weidner, Martin, Dawson, Meyer and See, Jones, and Weber.

Figs. 8 to 13 and Figs. 23 to 29, inclusive, and Fig. 49, are plottings of the percentage departures with auxiliary scales of coefficients. This method of plotting places the data a little more definitely on a precision basis, but is really only a slight departure from the type of diagrams used by Fteley and Stearns, Bazin, and Rehbock, in showing variations of coefficients with the heads for various heights of weir.

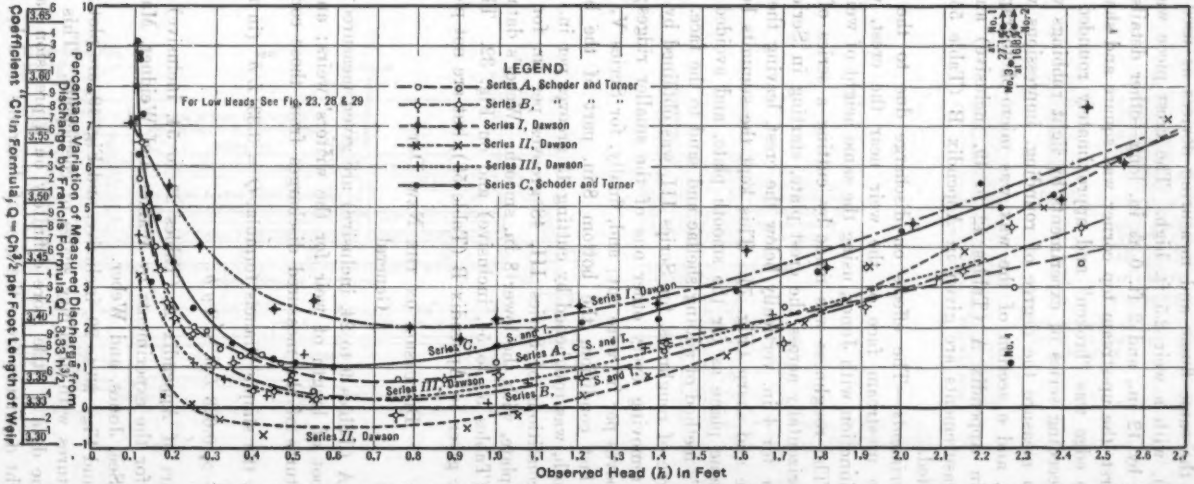

FIG. 8.—COMPARISON OF FLOW OVER SHARP OR NEARLY SHARP-CRESTED WEIRS WITHOUT END CONTRACTIONS. SIMILAR CONDITIONS FOR TESTS EXCEPT SLIGHT VARIATIONS IN SHARPNESS OF UP-STREAM TOP CORNER OF CREST, SMOOTHNESS OF UP-STREAM FACE, AND VERTICAL DISTRIBUTION OF VELOCITIES IN CHANNEL OF APPROACH.

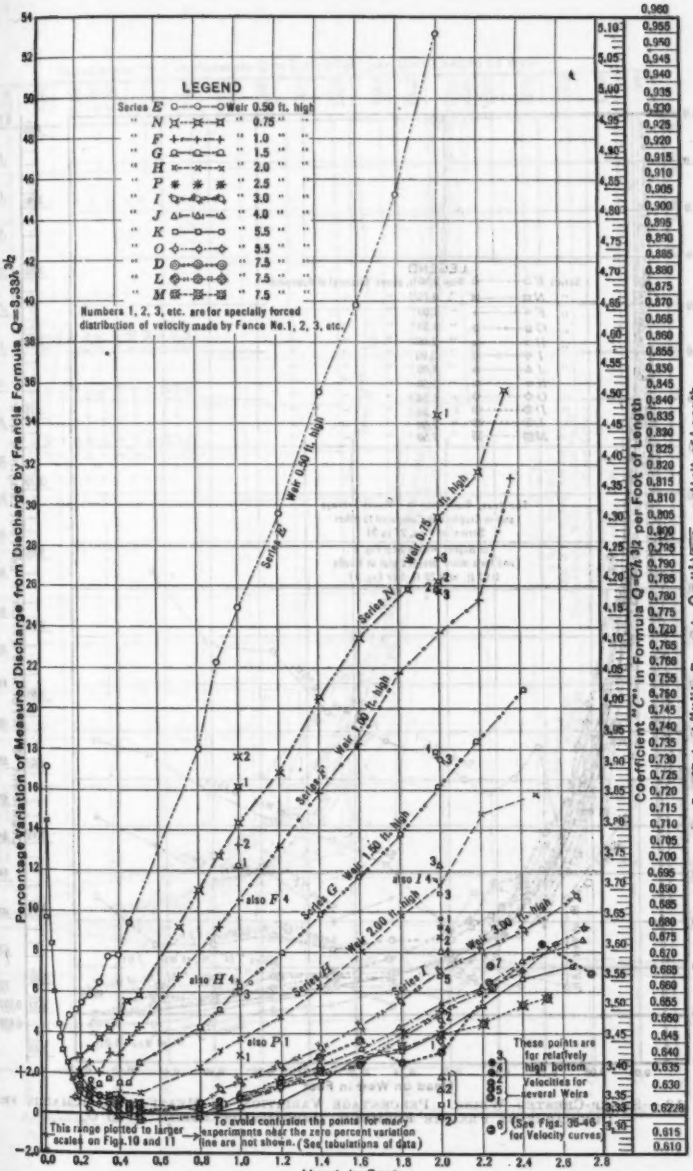

FIG. 9.—SHARP-CRESTED WEIRS. PERCENTAGE VARIATION OF MEASURED DISCHARGE FROM DISCHARGE BY FRANCIS FORMULA, SERIES D TO P.

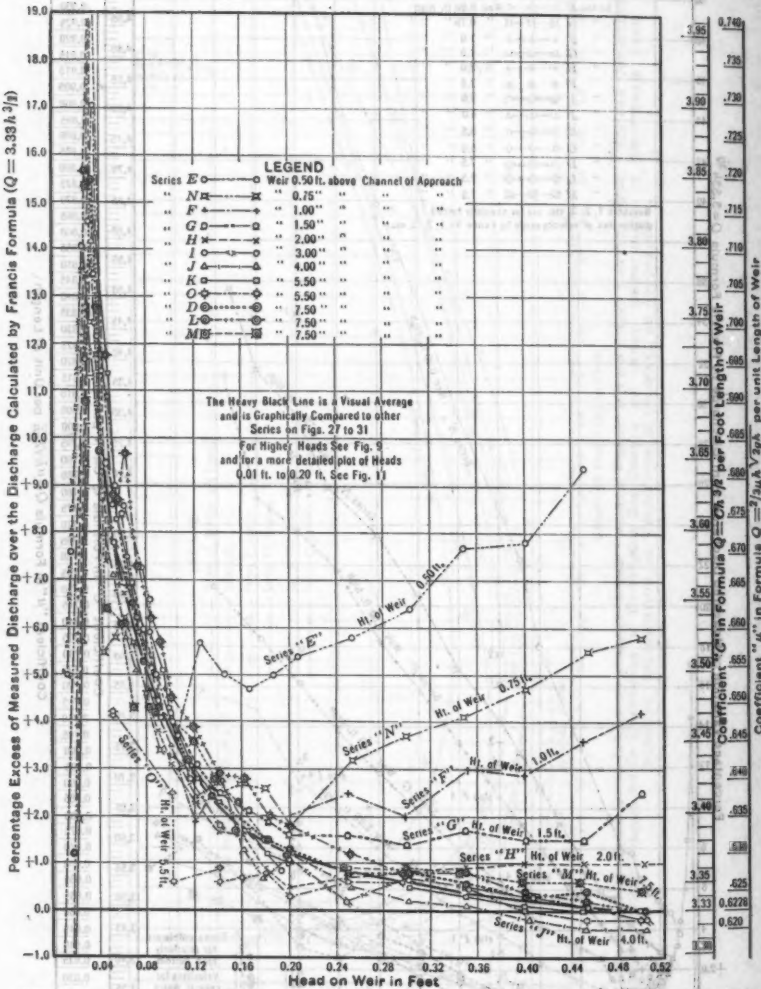

FIG. 10.—SHARP-CRESTED WEIRS. PERCENTAGE VARIATION OF MEASURED DISCHARGE FROM DISCHARGE BY FRANCIS FORMULA. LOW HEADS, SERIES D TO O.

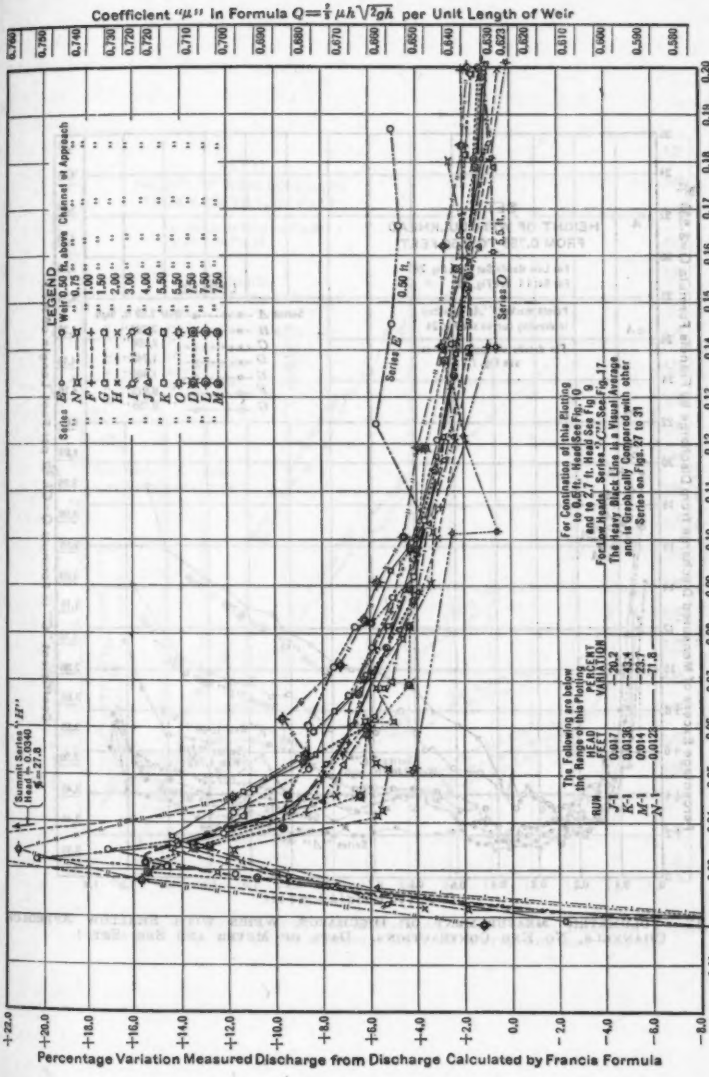

FIG. 11.—SHARP-CRESTED WEIRS. PERCENTAGE VARIATION OF MEASURED DISCHARGE FROM DISCHARGE BY FRANCIS FORMULA. EXTREMELY LOW HEADS, SERIES D TO N.

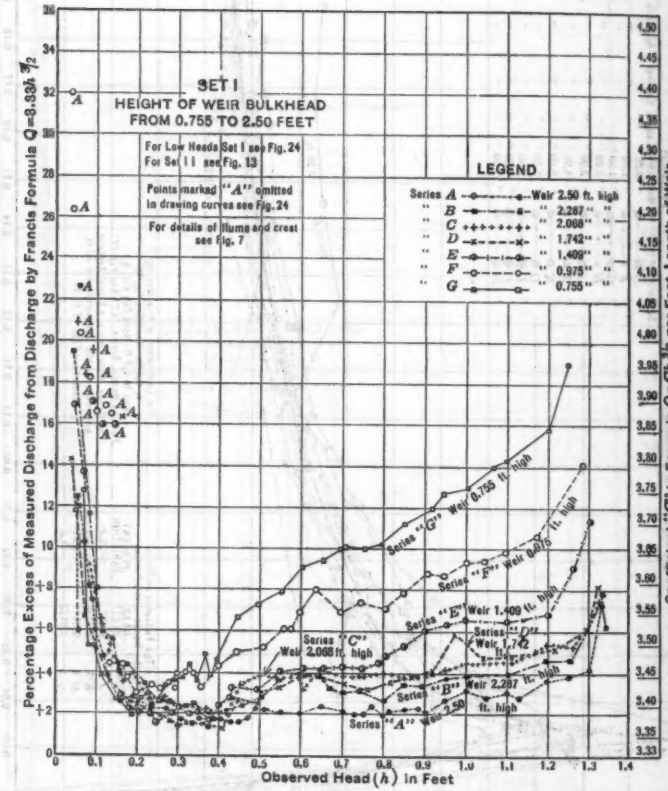

FIG. 12.—VOLUMETRIC MEASUREMENT OF DISCHARGE, WEIRS WITH SHALLOW APPROACH CHANNELS, NO END CONTRACTIONS. DATA OF MEYER AND SEE, SET I.

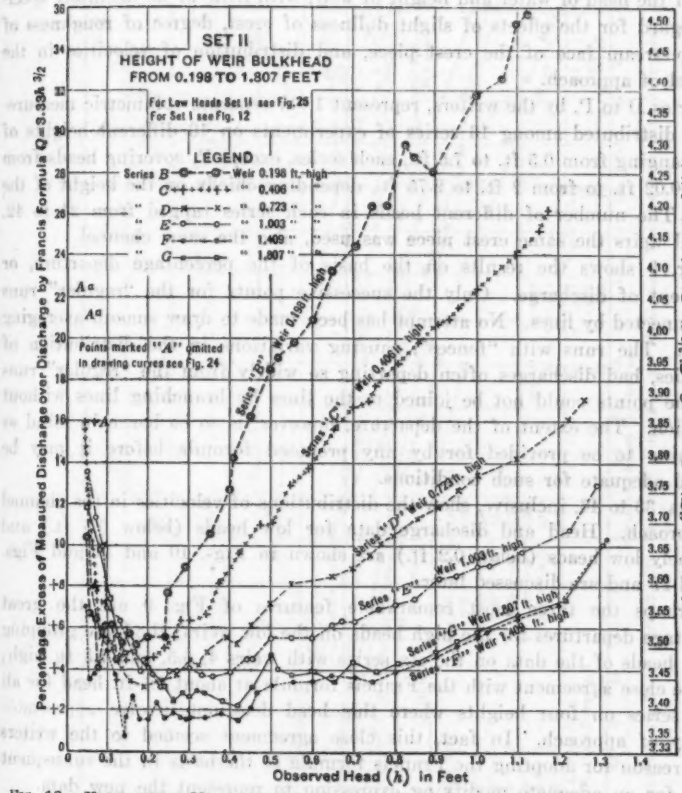

FIG. 13.—VOLUMETRIC MEASUREMENT OF DISCHARGE, WEIRS WITH SHALLOW APPROACH CHANNELS, NO END CONTRACTIONS. DATA OF MEYER AND SEE, SET II.

The Francis formula, as a tentative reference datum, was used for convenience in computations because of the availability of excellent tables* of discharges by this formula for heads differing by 0.001 ft. (interpolation to 0.0001 ft.).

New Data.—The writers' results yield thought-compelling illustrations of the magnitude of percentage variations in discharge. Particularly is this true if one adopts, for the moment, the usual custom of taking quantitative account only of the head of water and height of weir, with little or no definitely specified regard for the effects of slight dullness of crest, degree of roughness of the up-stream face of the crest piece, and distribution of velocities in the channel of approach.

Series D to P, by the writers, represent 1 017 separate volumetric measurements distributed among 13 series of experiments on 10 different heights of weir, ranging from 0.5 ft. to 7.5 ft., each series, except P, covering heads from about 0.02 ft. to from 2 ft. to 2.75 ft., depending chiefly on the height of the weir. The number of different heads in each series ranged from 24 to 42. For all weirs the same crest piece was used, also the same channel.

Fig. 9 shows the results on the basis of the percentage departure, or coefficient of discharge. Only the successive points for the "regular" runs are connected by lines. No attempt has been made to draw smooth averaging curves. The runs with "fences", causing variations in the distribution of velocities, had discharges often departing so widely from the "regular" runs that the points could not be joined to the lines by branching lines without confusion. The extent of the departure, however, is to be borne in mind as something to be provided for by any proposed formula before it may be deemed adequate for such conditions.

Figs. 33 to 46, inclusive, show the distributions of velocities in the channel of approach. Head and discharge data for low heads (below 0.5 ft.) and extremely low heads (below 0.2 ft.) are shown in Figs. 10 and 11 and Figs. 28 and 29 and are discussed later.

Perhaps the three most remarkable features of Fig. 9 are the great percentage departures for the high heads on the low weirs; the close grouping for all heads of the data on the six series with weirs 4, 5.5, and 7.5 ft. high; and the close agreement with the Francis formula at about 0.5-ft. head for all seven series on four heights where this head does not involve appreciable velocity of approach. In fact, this close agreement seemed to the writers ample reason for adopting the Francis formula as the basis in the subsequent search for an adequate modifying expression to represent the new data.

Old Formulas Tested by New Measurements

Figs. 14, 15, and 16 show in detail to what extent the discharges calculated for the measured heads by the formulas of Fteley and Stearns and Bazin, and the weir curves of Lyman, fail to agree with the measured discharges of the writers' "regular" runs of Series A to P. It is obvious that all these old formulas fail to fit even the general trend of the writers' "regular" runs.

* The tables by R. E. Horton, M. Am. Soc. C. E., in *Water Supply and Irrigation Paper No. 200*, U. S. Geological Survey, were used by the writers.

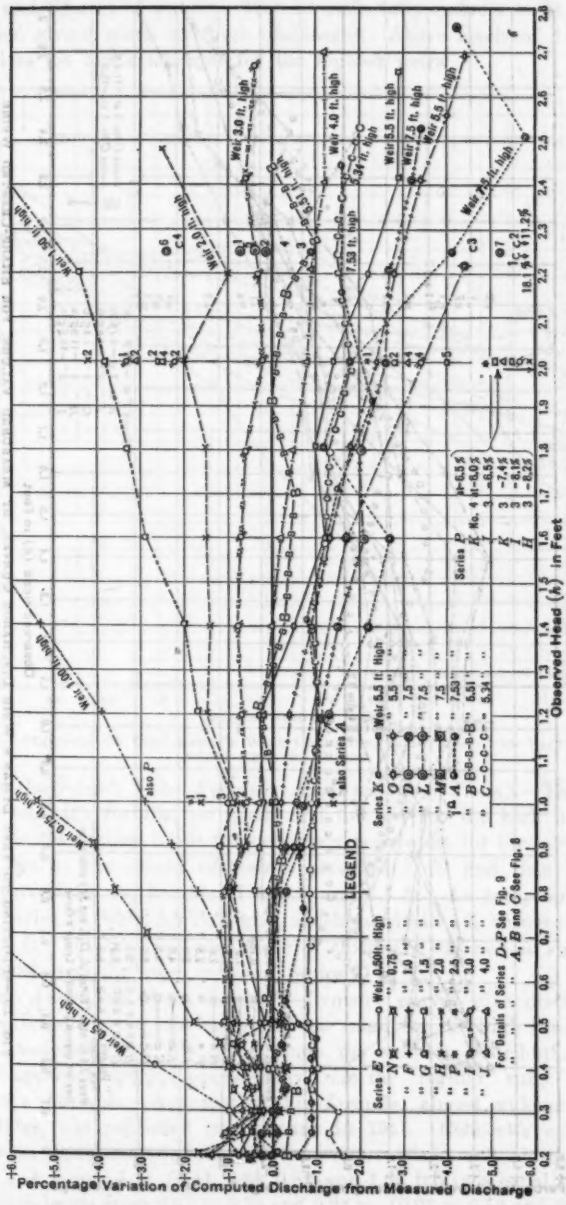

FIG. 14.—PLOTTING TO TEST FTELEY AND STEARNS' FORMULA BY MEASURED VALUES FOR SHARP-CRESTED WEIRS.

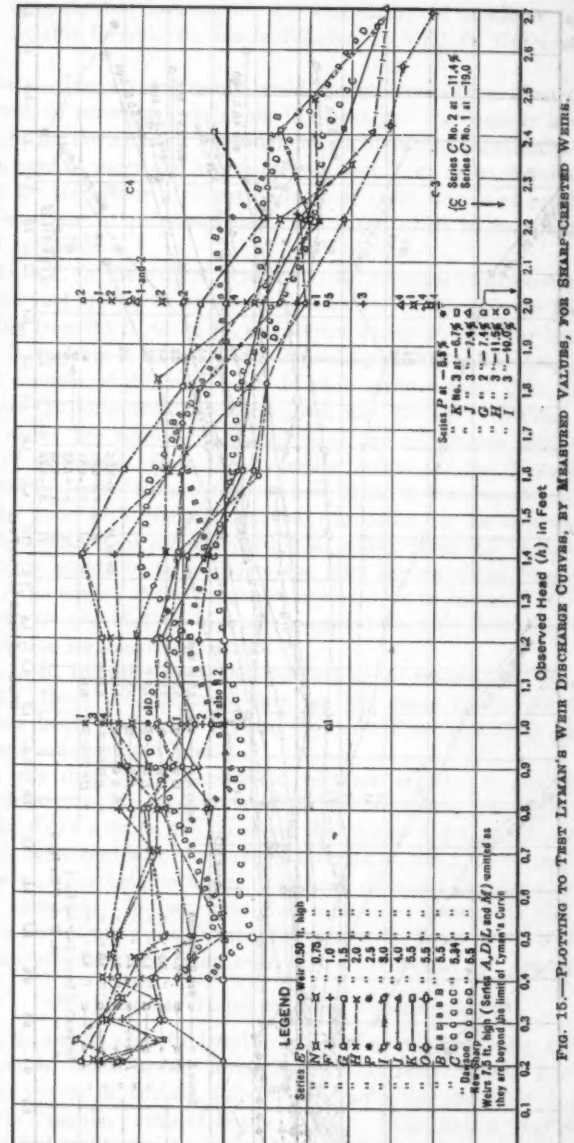

FIG. 15.—PLOTTING TO TEST LYMAN'S WEIR DISCHARGE CURVES, BY MEASURED VALUES, FOR SHARP-CRESTED WEIRS.

Fteley and Stearns' Formula.—This formula fails entirely to fit the four lowest weirs, giving much too high discharges. Above heads of 1.2 ft. the formula gives too low discharges for the highest weirs.

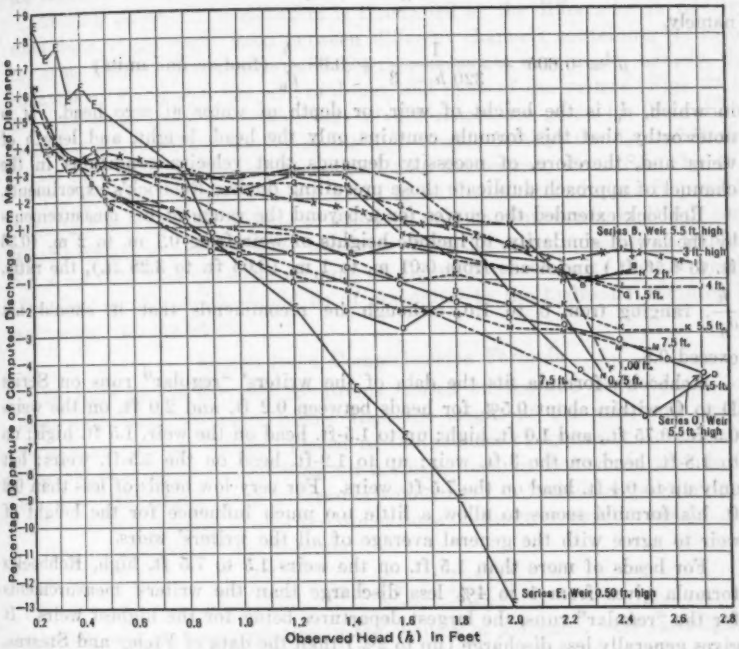

FIG. 16.—PLOTTING TO TEST BAZIN'S WEIR COEFFICIENTS BY MEASURED VALUES FOR SHARP-CRESTED WEIRS.

Bazin's Coefficients (and Formula Approximating Them).—These yield discharges too high for the low heads and too low for the high heads (see Fig. 16). At the highest heads the deficiency is greatest for the lowest weirs, becoming less as the height of weir approaches 3 ft. and then becoming greater progressively for heights of 4, 5.5, and 7.5 ft. An exception is to be noted for Series B (weir, 5.5 ft. high), which is within 1% between heads of 0.8 and 2.5 ft.; but the iron crest-piece of this weir was not as smooth nor as finely finished as the brass crests of Series D to P.

*Lyman's Averaging Weir Curves.**—Lyman's curves give discharges in general too large for the low heads and too small for the high heads. They are quite inadequate for the cases where the velocity distribution in the channel of approach differed considerably from the "regular" runs.

Rehbock's Formula.—Apparently, this formula, almost unknown in the United States, was published in Germany in 1911. Rehbock's experiments were made on sharp-crested, fully aerated weirs, without end contractions, of heights of $\frac{1}{8}$, $\frac{1}{4}$, $\frac{1}{2}$, and $\frac{1}{2}$ m. (0.41, 0.66, 0.82, and 1.64 ft.) in a canal $\frac{1}{2}$ m. (1.64 ft.) wide with heads from 0.01 to 0.18 and 0.23 m. (0.03 to 0.59 and 0.75 ft.).

* *Transactions*, Am. Soc. C. E., Vol. LXXVII (1914), Plate XXI, p. 1195.

In 1912 he published* his revised formula for μ in the equation,

$$Q = \frac{2}{3} \mu L h \sqrt{2 g h}$$

namely,

$$\mu = 0.605 + \frac{1}{320\, h - 3} + 0.08 \frac{h}{d_0} \text{ (foot-second units)}$$

in which, d_0 is the height of weir, or depth of water at zero head. It is noteworthy that this formula contains only the head, height, and length of weirs and, therefore, of necessity demands that velocity conditions in the channel of approach duplicate those prevailing during Rehbock's experiments.

Rehbock extended the curves for μ beyond the range of his measurements by the law of similarity, to include heights of weir from 0.1 m. to 2 m. (0.33 ft. to 6.56 ft.) and heads from 0.01 m. to 1 m. (0.03 ft. to 3.28 ft.), the ratio, $\frac{h}{d_0}$, ranging from 0 to 1.0, although he recommends that it should not exceed 0.8.

Rehbock's formula fits the data of the writers' "regular" runs on Series D to O within about 0.5% for heads between 0.2 ft. and 2.0 ft. on the weirs 0.5 ft., 0.75 ft., and 1.0 ft. high; up to 1.5-ft. head on the weir, 1.5 ft. high; up to 1.8-ft. head on the 3-ft. weir; up to 1.2-ft. head on the 5.5-ft. weirs; but only up to 0.4-ft. head on the 7.5-ft. weirs. For very low heads of less than 0.2 ft. his formula seems to allow a little too much influence for the height of weir to agree with the general average of all the writers' weirs.

For heads of more than 1.5 ft. on the weirs 1.5 to 7.5 ft. high, Rehbock's formula gives from 1 to 4% less discharge than the writers' measurements for the "regular" runs, the largest departures being for the highest weirs. It gives generally less discharge (up to 2%) than the data of Fteley and Stearns, Martin, and Meyer and See.

Considering, however, only those runs of the writers' tests with "fences" of such a construction as to cause a nearly uniform distribution of velocities from surface to bottom of the channel of approach (omitting most of the runs with highest heads on the highest weirs) Rehbock's formula gives closer agreement.

Comments.—Of course, if the same sort of limitations be imposed the older formulas would also appear to better advantage. This brings up the matter of choice (where choice is possible in using a weir or dam for precise measurements of discharge) between (1) making velocity conditions in the channel of approach agree with the specified conditions for which a particular formula applies; and (2) making the formula suitable for such variations in velocity conditions as may actually occur between the numerous different cases that arise in practice.

One phase of the situation is illustrated by the actual conditions of velocity distribution and discharge in the writers' Series A to P for which a fixed baffle system was used, the same for all heads on any one weir, and with seemingly small changes between the different weirs. These changes,

* Handbuch der Ingenieurwissenschaften, III Teil, Wasserbau, Band 2 (Stauwerke), Vierte Auflage, 1912, p. 59.

if any, were not in the main baffles shown on Fig. 1, but possibly in the angle made by small fender boards fastened between the main baffle-boards in order to bring about a distribution of velocities approximately symmetrical across the canal.

Another phase of the situation is illustrated by the differences in velocity conditions at the same head between different channels containing weirs of the same height. In some cases direct measurements were made; namely, the channels of Meyer and See, and of Jones, and those of the writers (Figs. 33 to 47). In other cases the indications of differences are given by the various trends of the coefficient curves at the high heads; namely, the data of Bazin, Fteley and Stearns, Martin, Meyer and See, the writers, etc. (See Fig. 49.)

The practical question would seem to be "to what extent is it possible to build a weir and its channel of approach of such dimensions, and with such a baffle system and stilling-raft, as to reproduce the velocity conditions implied for a formula that takes no account of differences in velocity distribution?" The data in this paper should be of assistance to the hydraulic engineer in answering this question.

Search for a Comprehensive Formula

In spite of the extreme conditions caused on the one hand by the low weirs with heads of water higher than the height of the weir, and on the other hand, by the "fences" placed in the channel of approach, considerable time has been spent in searching for a single formula that would provide for all cases. Of course, such a formula would have to include a velocity term, and could not contain merely the dimensions of the weir and the head.

The possibilities of the formula,

$$Q = 3.33\ L \left(h + a \frac{v^2}{2g}\right)^{\frac{3}{2}}$$

in which, v is the mean velocity in the channel of approach, were investigated in spite of the poor showing of the older formulas that follow this type or use it as a basis. It was thought that a suitable formula for a might be found. Consequently, the values of a were computed for all the runs, and plotted against various functions of the heads, heights of weir, ratios of head to height, ratios of head to total depth, etc. Note that this a is not the same as Bazin's α, which latter implies a variable μ (value of m for a weir of infinite height in $Q = m\ L\ h \sqrt{2\ g\ h}$).

There is a general tendency toward an increase in the value of a as the height of the weir increases, as found by earlier experimenters. Apparently, however, unless the distribution of velocities in the channel of approach be rigidly controlled (a matter of difficulty even for a laboratory weir with long approach), a simple law for a through a wide range of heights and heads is hardly to be expected.

Low Weirs with High Heads.—If only the five low weirs, 0.5 ft. to 2.0 ft. in height, of the writers' Series D to O be considered with only the high heads (head as great or greater than the height), the formula,

$$Q = 3.33\ L \left[h + (1.2 + 0.12\ d_0) \frac{v^2}{2g}\right]^{\frac{3}{2}} \text{ foot-second units} \ldots\ldots (A)$$

fits closely for the "regular" runs. In only 3 cases out of 50 is the calculated discharge more than 1% from the measured values.

The writers recognized that the influence of the distribution of velocities in the channel of approach to a weir needed experimental study.* Consequently, they made current-meter measurements of velocities a part of their program. It seemed reasonable to suppose that the chief assistance to the static head on a weir crest is that given by the velocity head of that part of the approaching water above the crest level. Therefore, the formula,

$$Q = 3.33\, L \left(h + \frac{v_a^2}{2g}\right)^{\frac{3}{2}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (B)$$

was tested by the new data. In this formula, v_a represents the true mean velocity in the channel of approach above the crest level, or, more strictly, $\frac{v_a^2}{2g}$ is the mean of the velocity heads above crest level, in case the difference between the two values is appreciable. The velocities are to be measured in the same cross-section as that in which the static head is measured.

Fig. 17 shows this formula tested by the new data, including all cases with specially forced velocity distributions by "fences" placed in the channel of approach. The formula fails chiefly, for the "regular" runs, by giving too low discharges for the weirs less than 3 ft. high when the head is greater than the height of weir. For the weirs more than 2.5 ft. high, including Series A, B, and C, all discharges, except 3 out of 126, are computed within 2% and the 3 exceptions are within 3 per cent. This includes all heads and all cases of abnormal velocity distributions for the weirs more than 2.5 ft. high.

For the five low weirs (Series E, N, F, G, and H), with heads higher than the weir, considered in an independent group, it appears that the formula,

$$Q = 3.33\, L \left[h + \left(1 + \frac{\sqrt{d_0}}{4}\right)\frac{v_a^2}{2g}\right]^{\frac{3}{2}} \text{ foot-second units} \ldots\ldots\ldots (C)$$

fits closely, but not so well as Equation (*A*).

After numerous trials with other functions of $\frac{v_a^2}{2g}$, it seemed obvious that the energy of that part of the approaching water below the crest level could not be disregarded. It seemed logical to assume that some function of $\frac{v_b^2}{2g}$ (v_b is the mean velocity in the channel of approach below the crest level) could be regarded as assisting the static head, but no suitable formula with a multiplier or other function of $\frac{v_b^2}{2g}$ within the same parenthesis as h and $\frac{v_a^2}{2g}$ was found. Perhaps the best tentative formula found was:

$$Q = 3.33\, L \left[\left(h + \frac{v_a^2}{2g}\right)^{\frac{3}{2}} + \frac{h}{3.33} \times \frac{v_b^2}{2g}\right] \text{ foot-second units} \ldots\ldots (D)$$

This formula seems to have no exact theory supporting it. However, the additive quantity, $\frac{h}{3.33} \times \frac{v_b^2}{2g}$, is at least in the nature of a volume.

* Some interesting quotations from hydraulic literature on this topic are given in Appendix C.

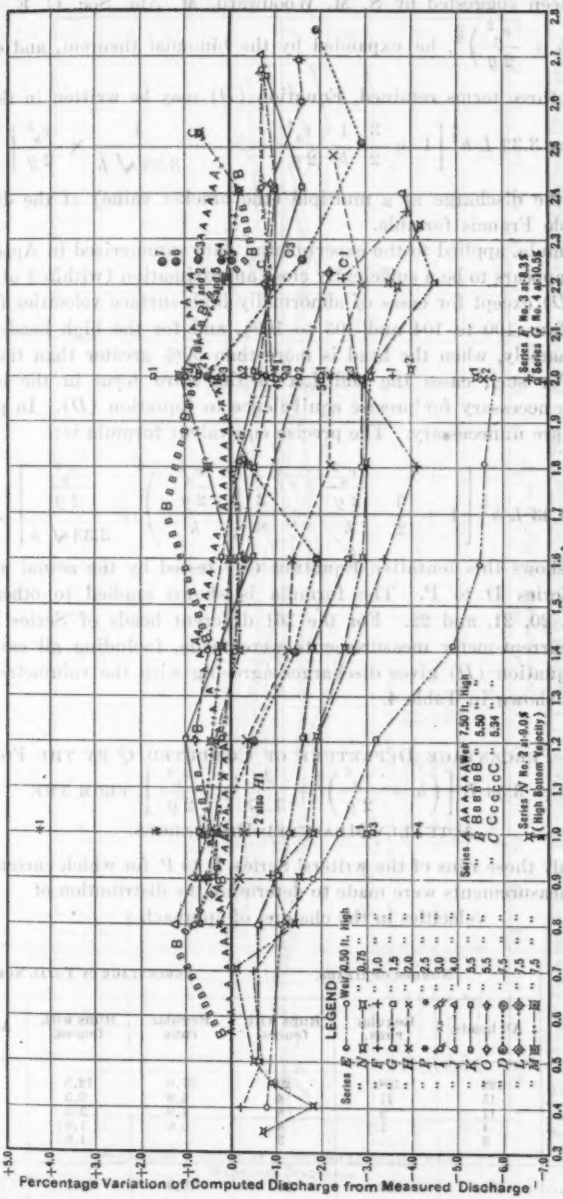

FIG. 17.—SHARP-CRESTED WEIRS. PERCENTAGE VARIATION OF DISCHARGE COMPUTED BY EQUATION (B) FROM MEASURED DISCHARGE.

It has been suggested by S. M. Woodward, M. Am. Soc. C. E., that, if the term, $\left(h + \frac{v_a^2}{2g}\right)^{\frac{3}{2}}$, be expanded by the binomial theorem, and only the first two or three terms retained, Equation (*D*) may be written in the form,

$$Q = 3.33\ L\ h^{\frac{3}{2}}\left[1 + \frac{3}{2}\ \frac{1}{h}\ \frac{v_a^2}{2g} + \ldots + \frac{1}{3.33\sqrt{h}} \times \frac{v_b^2}{2g}\right]$$

This gives the discharge as a multiple (the bracket value) of the discharge by the simple Francis formula.

This formula, applied to the experimental data summarized in Appendix B (Table 55) appears to be a sufficiently close approximation (within ¼ of 1%) to Equation (*D*) except for cases of abnormally high surface velocities (such as Series C, Runs 100 to 104 and 105 to 109), and for the high heads on the low weirs, namely, when the head is more than 50% greater than the height of weir. For such cases the addition of the third term in the binomial expansion is necessary for precise equivalence to Equation (*D*). In practice, more terms are unnecessary. The precise equivalent formula is:

$$Q = 3.33\ L\ h^{\frac{3}{2}}\left[1 + \frac{3}{2}\ \frac{\frac{v_a^2}{2g}}{h} + \frac{3}{8}\left(\frac{\frac{v_a^2}{2g}}{h}\right)^2 + \frac{\frac{v_b^2}{2g}}{3.33\sqrt{h}}\right] \ldots\ldots (E)$$

Fig. 18 shows this tentative Equation (*D*) tested by the actual measurements in Series D to P. The formula is shown applied to other series in Figs. 19, 20, 21, and 22. For the 161 different heads of Series D to P for which current-meter measurements were made, including all cases with "fences", Equation (*D*) gives discharges agreeing with the volumetric measurements as shown in Table 4.

TABLE 4.—PERCENTAGE DEPARTURE OF COMPUTED Q BY THE FORMULA, $Q = 3.33\ L\left[\left(h + \frac{v_a^2}{2g}\right)^{\frac{3}{2}} + \frac{h}{3.33} \times \frac{v_b^2}{2g}\right]$, FROM THE ACTUALLY MEASURED DISCHARGES.

(Includes only those runs of the writers' Series D to P for which current-meter measurements were made to determine the distribution of velocities in the channel of approach.)

Variation.	Number of Heads.			Percentage of Total Number.		
	All heads.	Regular runs.	Runs with fences.	Regular runs.	Runs with fences.	All heads.
1% or less	128	108	20	67.0	12.5	79.5
1-1.5%	15	11	4	6.8	2.5	9.3
1.5-2%	11	3	8	1.9	5.0	6.9
2.0-3%	4	1	3	0.6	1.9	2.5
3.0-5.3%	3	....	3	....	1.8	1.8
Total	161	123	38	76.3	23.7	100.0

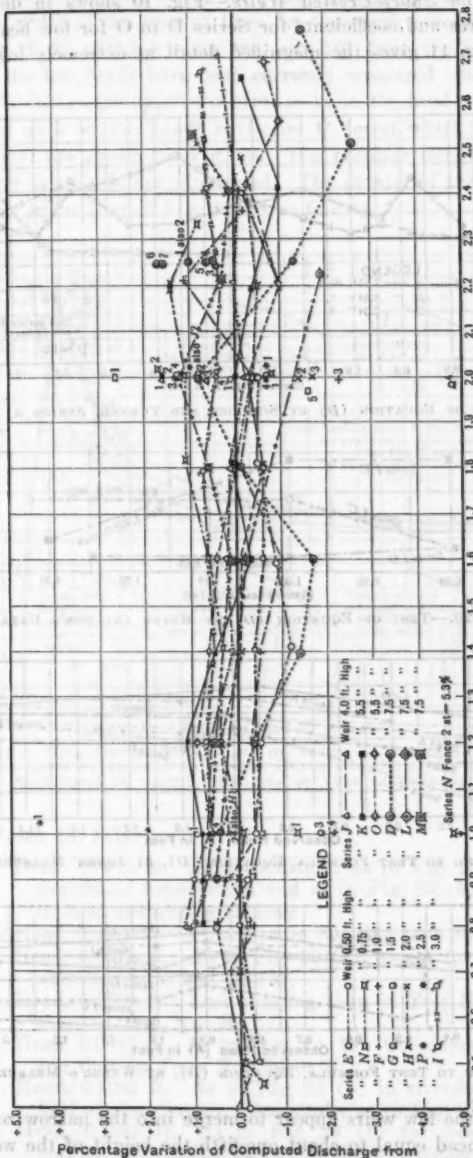

FIG. 18.—SHARP-CRESTED WEIRS. PERCENTAGE OF VARIATION OF DISCHARGE COMPUTED BY EQUATION (*D*) FROM MEASURED DISCHARGE.

Low Heads on Sharp-Crested Weirs.—Fig. 10 shows in detail the percentage departures and coefficients for Series D to O for low heads (less than 0.5 ft.), and Fig. 11 gives the magnified detail at extremely low heads (less than 0.2 ft.).

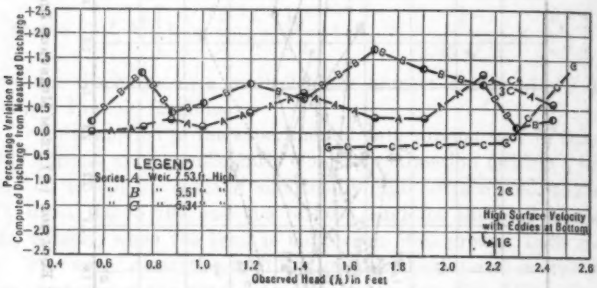

FIG. 19.—TEST OF EQUATION (*D*) BY SCHODER AND TURNER, SERIES A, B, AND C.

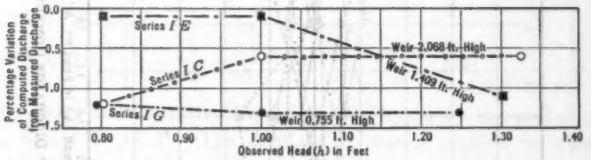

FIG. 20.—TEST OF EQUATION (*D*) BY MEYER AND SEE'S DATA.

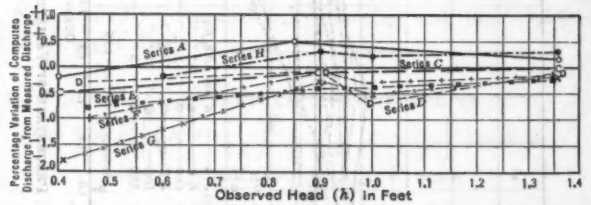

FIG. 21.—PLOTTING TO TEST FORMULA, EQUATION (*D*), BY JONES' MEASURED VALUES.

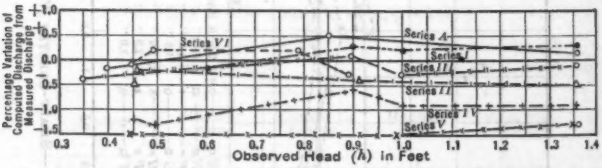

FIG. 22.—PLOTTING TO TEST FORMULA, EQUATION (*D*), BY WEBER'S MEASURED VALUES.

The lines for the low weirs appear to merge into the narrow zone common to all weirs at a head equal to about one-fifth the height of the weir.

In studying Figs. 10 and 11, the considerable effect of small errors in measurement is to be remembered; for example, an error of 0.0005 ft. in

measuring a head of 0.05 ft. causes an error of 1.5% in the computed discharge. If the width of the zone of lines for the various weirs covers 2% at 0.1 ft. head, or if the extreme edges are only 1% from the average line, it indicates that the low heads have been correctly measured within two-thirds of 1%, or 0.0007 ft. (assuming all the error to be in the head).

The writers' data on low heads in Series C (crest width = $\frac{3}{8}$ in., edge dulled by service) are shown on Fig. 23. The peculiar clinging phenomena at the extremely low heads are illustrated. The details of the conditions at the crest in the various tests of Fig. 23 are, as follows:

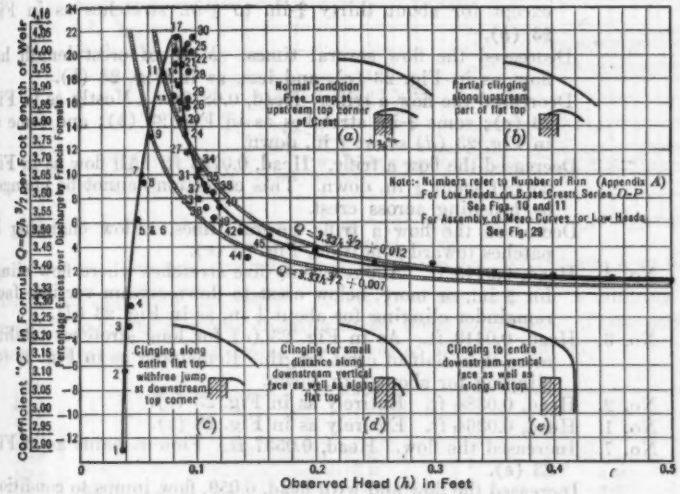

FIG. 23.—LOW-HEAD WEIR EXPERIMENTS, PART OF SERIES C, SHOWING DEPARTURES FROM DISCHARGE LAWS APPLICABLE TO HIGHER HEADS AND UNCERTAIN DISCHARGE CONDITIONS IN CRITICAL REGIONS OF VERY LOW HEADS.

Tests of March 16, 1914:

No. 24. Head, 0.0912 ft. Flow conditions as in Fig. 23 (*c*) for three-fourths of crest length and as in Fig. 23 (*b*) for remainder in local stretches $\frac{1}{2}$ in. long.

No. 20. Head, 0.0879 ft. A little more clinging than in Test No. 24.

No. 13. Head, 0.0813 ft. As in Fig. 23 (*c*), except for about ten $\frac{1}{2}$-in. stretches as in Fig. 23 (*b*).

No. 19. Head, 0.0878 ft. Less clinging than in Test No. 13, but more than in Test No. 20.

No. 26. Head, 0.0927 ft. As in Test No. 19. Considerable more clinging than in Tests Nos. 24 or 20.

No. 33. Head, 0.1020 ft. As in Fig. 23 (*b*) in stretches totaling one-half of crest length. Remainder as in Fig. 23 (*c*).

No. 36. Head, 0.1083 ft. As in Fig. 23 (*b*) for more than half of length. As in Fig. 23 (*c*) in short stretches $\frac{1}{4}$ in. long.

No. 39. Head, 0.1156 ft. As in Fig. 23 (*b*), except an occasional patch as in Fig. 23 (*c*).

Tests of March 17, 1914:

No. 28. Head, 0.0936 ft. Flow conditions as in Fig. 23 (*c*).

Increased flow a trifle. Head held 0.0035 ft. higher than in Test No. 28 as in Fig. 23 (*c*) for a few moments, but then broke into condition, Fig. 23 (*b*), in short stretches totaling more than one-fourth of crest length. Head then settled at 0.1037 ft., or 0.0101 ft. higher than in Test No. 28.

Decreased flow a trifle. Head, 0.099 ft. No change apparent.

Decreased flow a trifle. Head, 0.0955 ft. Flow as in Fig. 23 (*c*) for a little more than half of length, with remainder as in Fig. 23 (*b*).

No. 27. Decreased the flow a trifle. Head, 0.0934 ft. As in Fig. 23 (*c*) except for about thirty ¼-in. to ½-in. stretches as in Fig. 23 (*b*).

Decreased the flow several times. More of crest length has flow, as in Fig. 23 (*c*), and less, as in Fig. 23 (*b*).

Decreased the flow a trifle. Head, 0.0833 ft. Mostly as in Fig. 23 (*c*); some ¼-in. stretches as in Fig. 23 (*b*); and some as in Fig. 23 (*d*) scant ¼ in. down.

Decreased the flow a trifle. Head, 0.0793 ft. All flow as in Fig. 23 (*d*) scant ¼ in. down. This condition cannot be changed by brushing across crest.

Decreased the flow a trifle several times. Flow changing in patches toward condition, Fig. 23 (*e*).

No. 4. Head, 0.0445 ft. About twenty-five stretches where flow clings for ¾ in., or more, below crest to down-stream vertical face, remainder clinging for about ¼ in. as in Fig. 23 (*d*).

No. 3. Head, 0.0419 ft. As in Fig. 23 (*e*) for long stretches totaling more than half of crest length. Remainder as in Fig. 23 (*d*) clinging for about ¼ in. down.

No. 2. Head, 0.0384 ft. Entirely as in Fig. 23 (*e*).

No. 1. Head, 0.0364 ft. Entirely as in Fig. 23 (*e*).

No. 7. Increased the flow. Head, 0.0537 ft. Flow remains as in Fig. 23 (*e*).

Increased the flow and with head, 0.059, flow jumps to condition, Fig. 23 (*d*), clinging to vertical down-stream face for second ¼ in.

Increased the flow several times, but no further change.

No. 18. Head, 0.0853 ft. Flow as in Test No. 7.

Increased the flow several times and in patches it changes to condition, Fig. 23 (*c*), with head of 0.099 ft. Then, with head of 0.102 ft., flow changes to condition, Fig. 23 (*c*), for one-third of crest length and as in Fig. 23 (*b*) for remainder in forty-eight patches, ¼ in. to 1¾ in. long, and head settles at 0.1053 ft.

Increased the flow. Head, 0.118 ft. One-fifth of length as in Fig. 23 (*c*), remainder as in Fig. 23 (*b*).

Increased the flow. Head, 0.132 ft. As in Fig. 23 (*b*), except for some short patches as in Fig. 23 (*c*).

No. 44. Increased the flow. Head, 0.1438 ft. Flow jumps free as in Fig. 23 (*a*), except for $\frac{1}{16}$ in. to ⅛ in., where it clings to flat top as in Fig. 23 (*b*).

Increased the flow. Head, 0.168 ft. As before, but clings to flat top for $\frac{1}{16}$ to ⅛ in. only in spots.

Increased the flow. Head, 0.196 ft. Conditions as before (for 0.168-ft. head).

Similar data on low heads, down to about 0.04 ft., from Meyer and See's experiments, are shown on Figs. 24 and 25. On these diagrams, the points marked *A* represent measurements from a series of progressively increased low

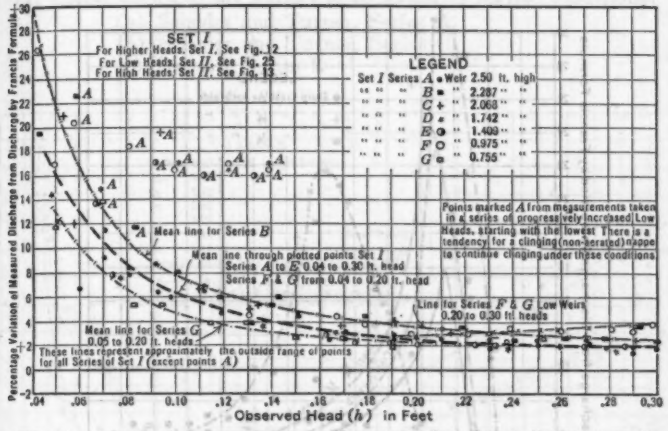

FIG. 24.—LOW-HEAD EXPERIMENTS BY MEYER AND SEE, SET I, SERIES A TO G.

heads, starting with the lowest. There is a tendency for a clinging (non-aerated) nappe to continue clinging under these conditions. These points, *A*, were not considered in drawing the curves of Figs. 24 and 25.

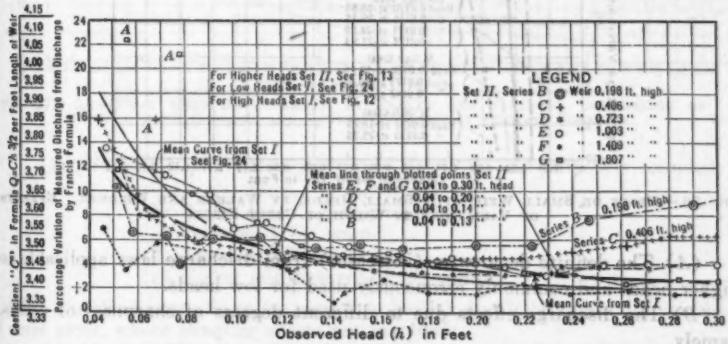

FIG. 25.—LOW-HEAD EXPERIMENTS BY MEYER AND SEE, SET II, SERIES B TO G.

Data from Walker and Weidner's experiments at low heads on several widths of crest piece are shown on Fig. 26. Fig. 27 shows Fteley and Stearns' observed data* for low heads and the basis for the averaging curves used in other plottings. Fig. 28 is a partial assembly of low-head data, showing the observed values of Bazin in comparison with those of others. The crests ranged from known definitely sharp conditions through known dulled corners and uncertain cases to known definitely rounded corners.

* *Transactions*, Am. Soc. C. E., Vol. XII (1883), pp. 25–28.

Fig. 29 is a general assembly of curves representing all the low-head data on crests with sharp, dull, uncertain, and slightly rounded, top, up-stream, corners. It shows:

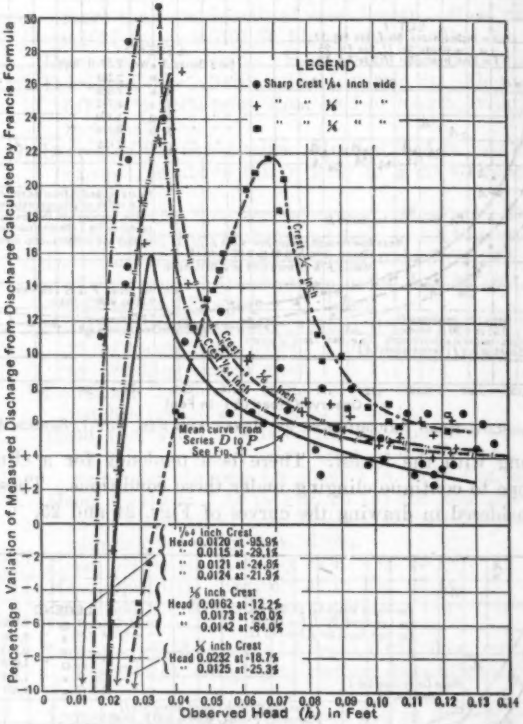

FIG. 26.—TESTS ON SMALL WEIR WITH SMALL HEADS, BY WALKER AND WEIDNER. EFFECTS OF VARIATIONS IN WIDTH OF CREST PIECE.

(*A*) The definite departures (increases) from discharge laws applicable to higher heads and commonly erroneously used for low heads.

(*B*) The discharge effects due to different degrees of sharpness of crests, namely,

(1) Crests generally accepted as strictly sharp and so described by experimenters:

(*a*) Francis.
(*b*) Fteley and Stearns.

(2) Crests generally accepted as strictly sharp, but with experimenter's description incomplete and indefinite:

(*a*) Bazin.

(3) Crests known to the writers to be strictly sharp:

(*a*) Schoder and Turner, Series D to P.
(*b*) Schoder and Turner, Series B.

(*c*) Meyer and See, Series I and II.
(*d*) J. O. Jones.
(*e*) Walker and Weidner.

(4) Crests known to the writers to be dulled by service conditions:
(*a*) Schoder and Turner, Series A.
(*b*) Schoder and Turner, Series C.
(*c*) Dawson, Series I.

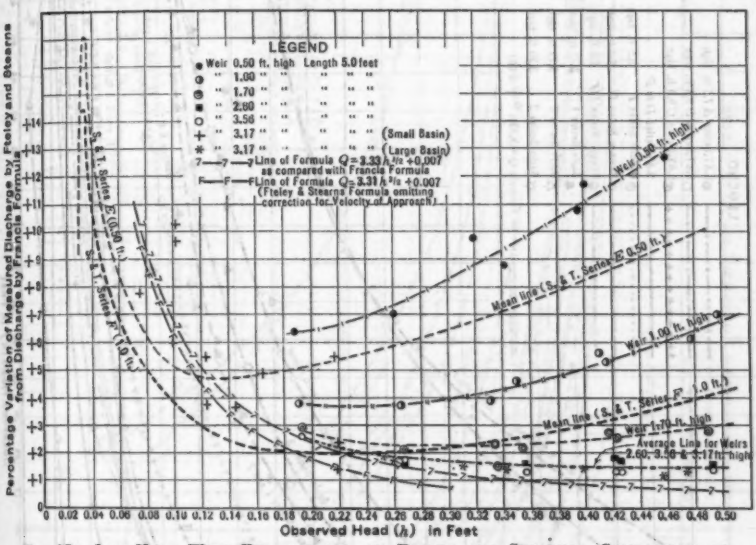

FIG. 27.—LOW-HEAD WEIR EXPERIMENTS, BY FTELEY AND STEARNS. COMPARISON OF RESULTS WITH SCHODER AND TURNER ON BASIS OF FRANCIS FORMULA.

(5) Crests known to the writers to be definitely and purposely rounded:
(*a*) J. O. Jones, $\frac{1}{64}$ in., round.
(*b*) J. O. Jones, $\frac{1}{24}$ in., round.
(*c*) Dawson, $\frac{1}{8}$ in., round.
(*d*) Dawson, $\frac{1}{4}$ in., round.
(*e*) Fteley and Stearns, $\frac{1}{4}$ in., round.

(*C*) The discharge effects (at very low heads) due to different top widths of crest piece, where clinging effects are involved:
(*a*) Walker and Weidner, $\frac{1}{64}$ in. wide.
(*b*) Walker and Weidner, $\frac{1}{8}$ in. wide.
(*c*) Walker and Weidner, $\frac{1}{4}$ in. wide.
(*d*) Schoder and Turner, Series D to P, $\frac{3}{32}$ in. wide.
(*e*) Schoder and Turner, Series B, $\frac{3}{8}$ in. wide.
(*f*) Schoder and Turner, Series C, $\frac{3}{8}$ in. wide.

All these new data show a general agreement with the findings of earlier experimenters that there is an increasing percentage increase in discharge as the head decreases below about 0.4 ft. for sharp-crested weirs having heights of more than 2 ft. The law of increase in discharge according to Fteley and

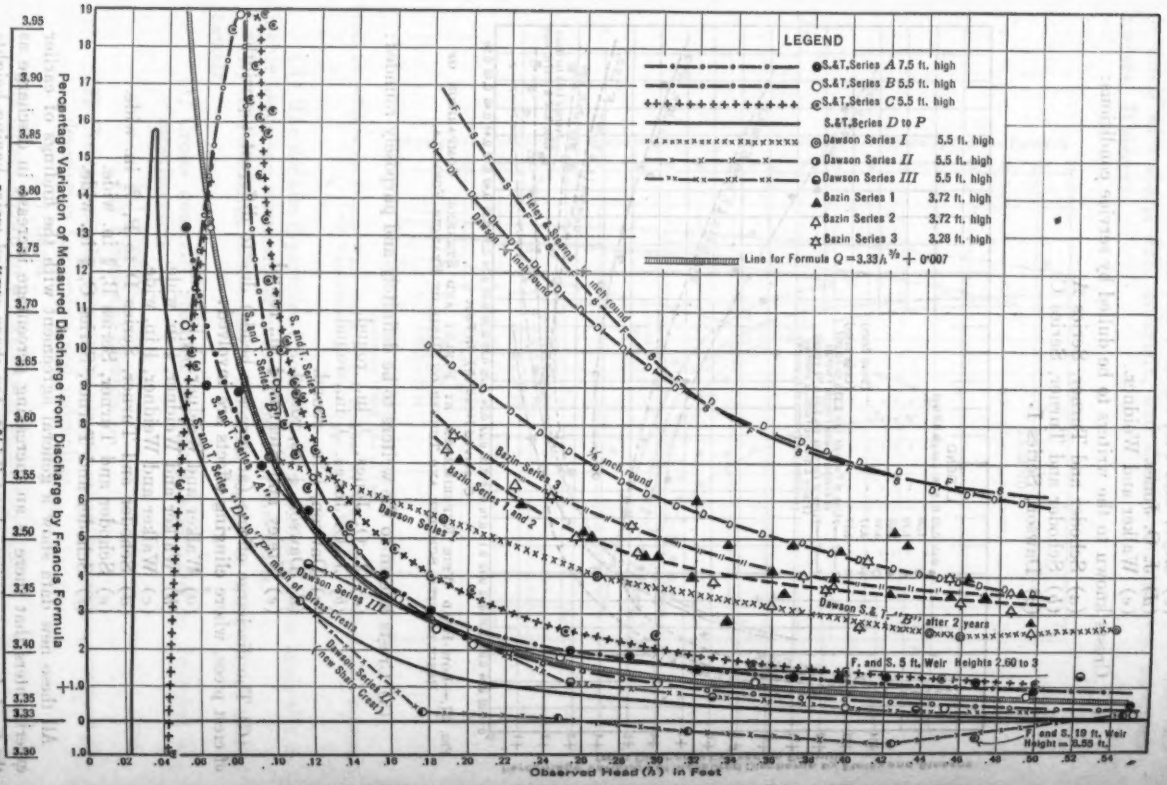

FIG. 28.—LOW-HEAD WEIR EXPERIMENTS, SHOWING VALUES OBTAINED BY VARIOUS EXPERIMENTERS.

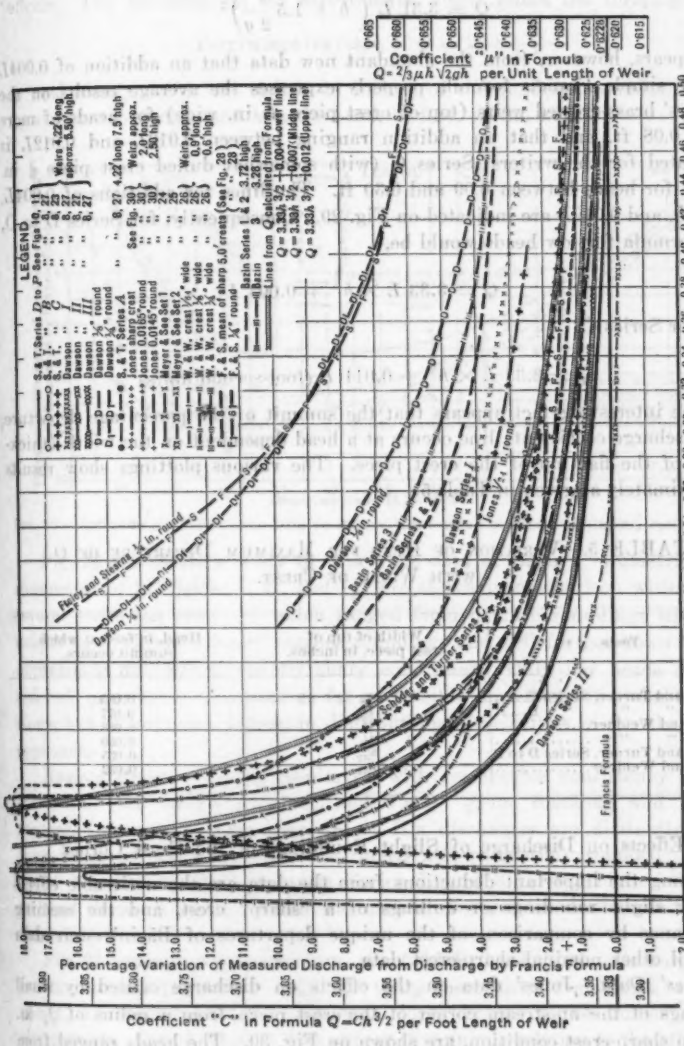

FIG. 29.—LOW-HEAD WEIR EXPERIMENTS. ASSEMBLY OF MEAN CURVES.

FIG. 28.—LOW-HEAD WEIR EXPERIMENTS, SHOWING VALUES OBTAINED BY VARIOUS EXPERIMENTERS.

Stearns is expressed by adding $0.007L$ (foot-second units) to their basic expression,

$$Q = 3.31\ L\left(h + 1.5\frac{v^2}{2g}\right)^{\frac{3}{2}}$$

It appears, however, from the abundant new data that an addition of $0.004L$ to the simple Francis formula properly expresses the average results on the writers' brass-crested weirs (top of crest piece $\frac{3}{32}$ in. wide) for heads of more than 0.08 ft. and that an addition ranging between $0.010L$ and $0.012L$ is indicated for the writers' Series C (with somewhat dulled crest piece $\frac{1}{8}$ in. wide) for heads between 0.09 and 0.30 ft. The lines for additions of $0.004L$, $0.007L$, and $0.012L$ are indicated on Fig. 29. Consequently, for Series D to O, the formula for low heads would be,

$$Q = 3.33\ L \times h^{\frac{3}{2}} + 0.004\ L$$

and for Series C,

$$Q = 3.33\ L \times h^{\frac{3}{2}} + 0.011\ L \text{ (foot-second units)}$$

The interesting fact appears that the summit of the percentage departure (or discharge coefficient) line occurs at a head dependent on the width (thickness) of the flat top of the crest piece. The various plottings show results approximately as given in Table 5.

TABLE 5.—VARIATION OF HEAD FOR MAXIMUM DEPARTURE OF Q, WITH WIDTH OF CREST.

Tests.	Width of top of crest piece, in inches.	Head, in feet, at which summit occurs.
Schoder and Turner, Series C.....	$\frac{3}{8}$	0.085
" " " B.....	$\frac{11}{32}$	0.077
Walker and Weidner.....	$\frac{1}{4}$	0.068
" " "	$\frac{1}{8}$	0.039
Schoder and Turner, Series D to P	$\frac{3}{32}$	0.035
Walker and Weidner.....	$\frac{1}{64}$	0.032

Effects on Discharge of Slight Roundings of the Crest Corner

Among the important deductions from the data are the noticeable effects of very slight roundings or dullings of a "sharp" crest, and the seeming significance by comparison, of the unique departures of Bazin's weir data from all other nominal sharp-crest data.

Jones' Data.—Jones' data on the effects on discharge caused by small roundings of the up-stream corner of the crest piece, from a radius of $\frac{1}{4}$ in. down to sharp-crest condition, are shown on Fig. 30. The heads ranged from 0.18 ft. to 1.36 ft. on a weir 2.5 ft. high and 2 ft. long. The percentage increase in discharge, due to rounding, over sharp-crest values is seen to decrease with the head. It amounts to 1.6% for the $\frac{1}{24}$-in. radius rounding at 0.5-ft. head, and to 0.7% at 1.0-ft. head. A rounding with $\frac{1}{60}$-in. radius

causes an increase of 0.85% at 0.5-ft. head and 0.45% at 1-ft. head. Still smaller roundings with less than $\frac{1}{100}$-in. radius, cause definitely measurable effects. For the range of his experiments, Jones found the formula,

$$\text{Percentage increase in } Q = 22.8 \frac{R}{h^{\frac{3}{4}}}$$

to hold closely. R is the radius, in inches, of the quadrant rounding of the up-stream top corner, and h is the head, in feet, on the weir.

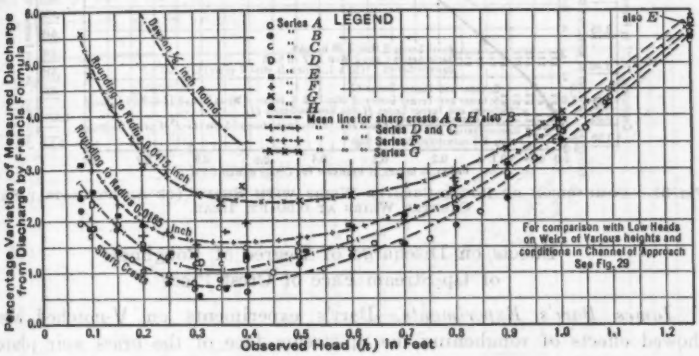

FIG. 30.—EFFECTS ON DISCHARGE DUE TO SMALL ROUNDINGS OF EDGE OF CREST, JONES' DATA.

Fig. 28 represents a study in which the data of Bazin's volumetric measurements* of his Series I, II, and III are shown in comparison with data of several weirs the crests of which ranged from definite roundings with $\frac{1}{4}$-in. radius down through Jones' smaller roundings and still further down through appreciable dullings to strictly sharp conditions. Only low heads are considered, in order to eliminate as far as possible the effects of probable variations, but of unknown degree, in distributions of velocities in the channels of approach.

Bazin's Results.—These results lie with remarkable consistency between Dawson's $\frac{1}{8}$-in. radius rounding and Jones' $\frac{1}{24}$-in. rounding and are far separated from the sharp-crest data. His discharges are distinctly higher than those of crests known to be dulled and perceptibly rounded; namely, about 1% higher than Dawson's Series I, and 3 to 4% higher than the writers' Series A and C, which themselves have greater discharges than the sharp crests.

Fig. 31 represents a comparison, all at 0.5-ft. head, of the discharges over crests rounded to various radii for weirs more than 3 ft. high, and also shows the discharges for crests with edges known to be dulled and for sharp crests. Again, Bazin's data appear in a unique relation to the other data.

* Bazin described his crest piece as an iron plate "bien dressée," 0.007 m. (0.276 in.) thick. The "bien dressée" is translated by Marichal and Trautwine in *Proceedings*, Engineers Club of Philadelphia, January, 1890, as "carefully straightened." The drawing, Fig. 9, Plate 2, in *Annales des Ponts et Chaussées*, October, 1888, shows the plate to be about 7.4 in. wide, with top edge square and flat. The title reads, "Section of the crest of the weir of sharp-edged type" ("type à arête vive").

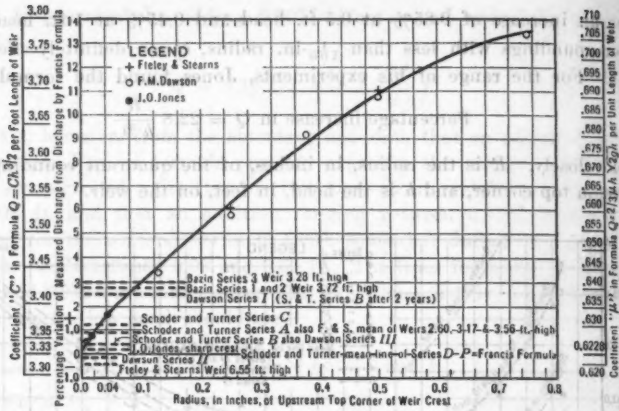

FIG. 31.—COMPARISON OF ROUND-CRESTED WEIRS WITH SHARP (OR APPROXIMATELY SHARP) CRESTED WEIRS AT 0.50-FT. HEAD.

Effects on Discharge of Degree of Roughness of Up-Stream Face of Crest Piece

James Barr's Experiments.—Barr's experiments on V-notched weirs* showed effects of roughening the up-stream face of the brass weir plate as given in Table 6.

TABLE 6.—RELATION OF DISCHARGE TO ROUGHNESS OF UP-STREAM FACE OF WEIR.

Condition of up-stream surface of notch.	PERCENTAGE INCREASE IN DISCHARGE OVER VALUE FOR PLAIN SURFACE.	
	Head = 8 in.	Head = 4 in.
Plain surface	0.	0.
One coat varnish (shellac)	0.39	0.26
Coated with medium emery	1.8	1.2
Coated with coarse emery	2.4	1.7

Weber's Data.—These data are represented on Fig. 32. The percentage increases in discharge, due to roughening the up-stream face, do not seem to decrease as rapidly with the increase of the head as for the slight roundings of the crest edge. Thus, Series V at 0.5-ft. head shows 1.85% increase over smooth-plate conditions, and even at 1.35-ft. head the increase is still 1.2 per cent.

The part of the plate near the crest appears to have more influence than that farther removed, although the addition of 8 in. of rough surface below the top 4 in. caused a further increase of discharge averaging fully 0.5% for heads between 0.3 ft. and 1.35 ft.

* *Engineering* (London), April 8 and 15, 1910.

Bazin's Discharges Too High for Sharp-Crest and Smooth-Plate Condition

Again returning to Bazin's data (see Table 7), it appears that a combination of slight rounding at the edge and rusting of the up-stream face could have produced just such a percentage effect as Bazin's measurements show when compared with other sharp crests. In fact, Dawson's Series I as compared with the writers' Series B, using the same weir and crest piece, shows nearly the same increase of discharge as Bazin's departure from all the other known sharp crests.

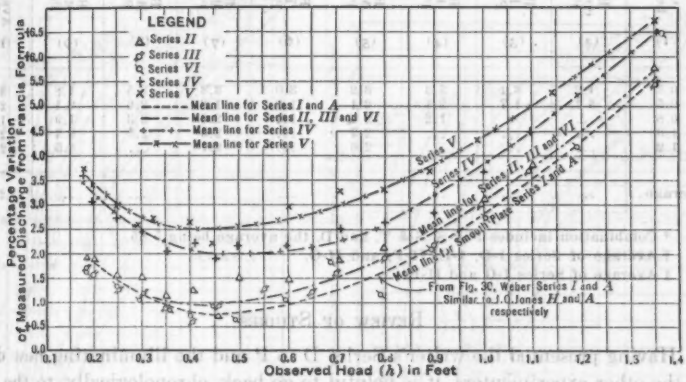

FIG. 32.—RELATION OF DISCHARGE TO ROUGHNESS OF UP-STREAM FACE OF CREST PIECE. WEBER'S DATA.

The writers see no reason for questioning Bazin's measurements of heads and discharges in the ratings of his "standard weir" by Series I, II, and III. The fact that he did not divert an already established flow into his measuring basin, but started with a static head, seems to be amply offset by the long duration of his measurements, from 10 to 20 min. Consequently, there is no implied criticism of the great mass of subsequent measurements made by Bazin, using his "standard weir" as a reference except as this may have deteriorated. The only point suggested is that his data on Series I to X do not appear to belong in the sharp-crest, smooth-plate category.

Rehbock* (and Hansen† before him), has supplied experimental evidence showing that Bazin's discharges are too high as compared to their values for sharp-crested weirs. Bazin himself tabulated‡ the excesses of his values over those of Fteley and Stearns for both the latter's 1877 and 1879 experiments.

It should be mentioned that Gardner S. Williams, M. Am. Soc. C. E., reached the conclusion long ago§ that the data of Francis and of Fteley and Stearns were more reliable than those of Bazin.

* Handbuch der Ingenieurwissenschaften, III Teil, Wasserbau, Band 2 (Stauwerke), Vierte Auflage, 1912, p. 59.

† *Zeitschrift* des Vereins deutscher Ingenieure, Band 36, p. 1057.

‡ *Annales des Ponts et Chaussées*, October, 1888; also, *Proceedings*, Engineers' Club of Philadelphia, January, 1890, p. 279.

§ *Transactions*, Am. Soc. C. E., Vol. XLIV (1900), p. 321.

TABLE 7.—BAZIN'S PERCENTAGE EXCESS DISCHARGE, BY HIS COEFFICIENTS, OVER THE EXPERIMENTAL DATA (OLD AND NEW) BY VARIOUS OBSERVERS.

(In the headings, B, 3.72 means Bazin's weir, 3.72 ft. high; F & S = Fteley and Stearns; D = Dawson; M & S = Meyer and See; S & T = Schoder and Turner. All nominally sharp crests.)

Observed head, in feet.	B, 3.73 over combination.* Percentage.	B, 3.72 over F & S, 3.17. Percentage.	B, 2.46 over F & S, 2.60. Percentage.	B, 2.46 over M & S, 2.50. Percentage.	B, 1.64 over F & S, 1.70. Percentage.	B, 1.64 over M & S, 1.60.† Percentage.	B, 1.16 over Martin, 1.20. Percentage.	B, 0.79 over M & S, 0.74.‡ Percentage.	Average excess. Percentage.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
0.3	4.2	3.1	3.3	3.2	3.0	3.3	3.5	1.8	3.2
0.5	3.1	1.7	2.1	2.1	1.9	1.8	2.6	2.1	2.2
0.8	...	0	1.2	2.4	1.1	2.6	2.3	1.9	1.9
1.0	...	...	...	2.7	...	2.5	2.5	3.3	2.7
1.2	...	...	...	2.6	...	2.7	...	3.5	2.9
Average..	...	...	...	...	...	...	...	...	2.6

* Combination includes F & S, S & T, and D, the average being 5.85.
† Average of Series I-D, I-E, II-F, and II-G.
‡ Average of Series I-G and II-E.

REVIEW OF STUDIES

Having presented the writer's Series D to P and the illuminating new data of the other experimenters, it is helpful to go back, chronologically, to the situation that confronted the writers and, to some extent, the engineering public since the publication of Bazin's volumetric measurements in 1888.

Among the preliminary studies made by the writers, three comparisons may be of some interest.

First Comparison: Six Weirs in Same Channel.—Fig. 8 shows results on a group of six weirs set in the same canal and for which general conditions seemed nearly the same. Among them one might expect close agreement in discharges for the same heads. The averaging lines for the several series show among themselves extreme variations for all heads from 2 to 4 per cent. This is for the "regular" experiments; that is, those made with the regular deflector baffles and stilling-raft arranged to secure a minimum of irregularity in velocities in the channel of approach.

These variations in discharges seem to be rather less than might have been expected if different channels of approach had been used involving the judgments of different men as to sufficiency of baffle systems and proper fineness of finish of crest pieces for correct designation as "sharp-crested."

The reason for these variations in discharges might lie (1) in the water; or (2) in the weir. In detail, assuming no challenging question as to the general arrangement, dimensions, heads, and discharges, and relying on recent precise investigations, such as Rehbock's, as assurance that effects of temperature changes in the water are exceedingly small at heads above the very lowest, the reasons might lie (*a*) in velocity conditions in the channel

of approach; or (*b*) in the mechanical conditions at the weir, particularly the sharpness of crest and degree of roughness of the up-stream face.

(*a*).—Examining the distribution of velocities for Series A, B, and C (these are shown on Figs. 33 to 35, and the velocity curves from other series as Figs. 36 to 48, inclusive (see, also, Appendix B (Table 55)), it is seen that the high weir of Series A has an average ratio of about 1.74 between the mean velocity above the crest level and the mean velocity in the whole channel of approach, as compared with an average ratio of about 1.38 for Series B and 1.32 for Series C (the "regular" runs).

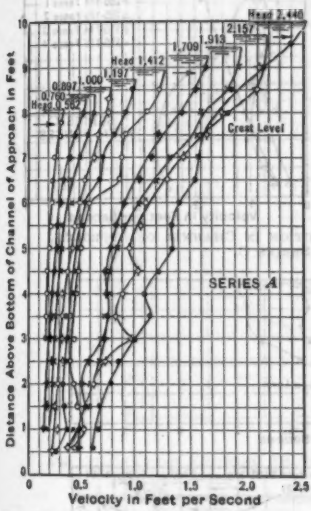

FIG. 33.—CURVES SHOWING VELOCITY DISTRIBUTION IN CHANNEL OF APPROACH, SCHODER AND TURNER, SERIES A.

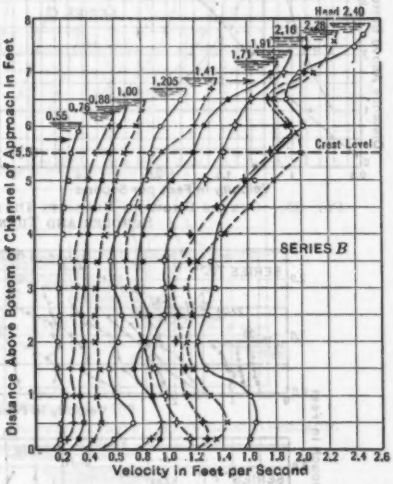

FIG. 34.—CURVES SHOWING VELOCITY DISTRIBUTION IN CHANNEL OF APPROACH, SCHODER AND TURNER, SERIES B.

The mean velocities above the crest level, when plotted against the heads, show practically one line for the three series, whereas one would expect somewhat lower velocities for the deeper channel of Series A. (No velocity measurements were taken for Dawson's experiments, but the channel conditions were the same as in Series B.)

By the old formulas, which correct for the velocity of approach, such as those of Fteley and Stearns, or Bazin, a weir 7.5 ft. high would show less discharge than one 5.5 ft. high by an amount about 1.6% for 2.5-ft. head, 0.7% for 1.5-ft. head, 0.4% for 1-ft. head, and 0.1% for 0.5-ft. head. The experiments in Fig. 8 show a decrease only for the highest head as compared with the weirs 5.5 ft. high. This may be due, in small part, to the obstruction offered by the deflector hood to the formation of bottom velocities of normal free-channel magnitude, causing, in effect, flow conditions similar to those for a weir less than 7.5 ft. high in a long canal free from obstructions at the bottom for a much longer distance than 25 ft. up stream.

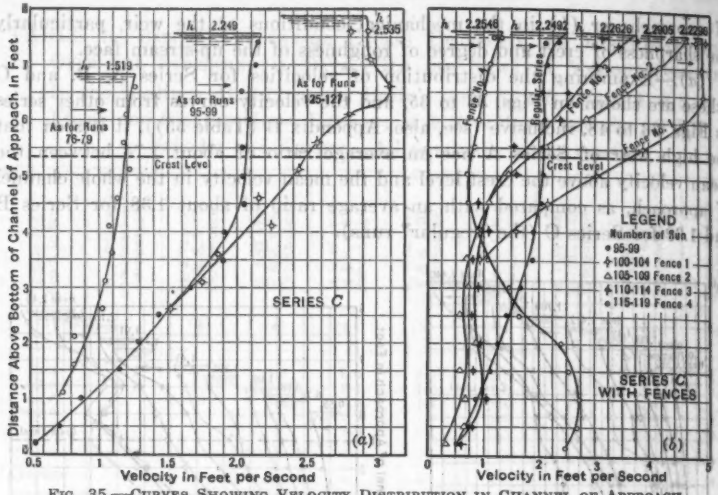

FIG. 35.—CURVES SHOWING VELOCITY DISTRIBUTION IN CHANNEL OF APPROACH, SCHODER AND TURNER, SERIES C.

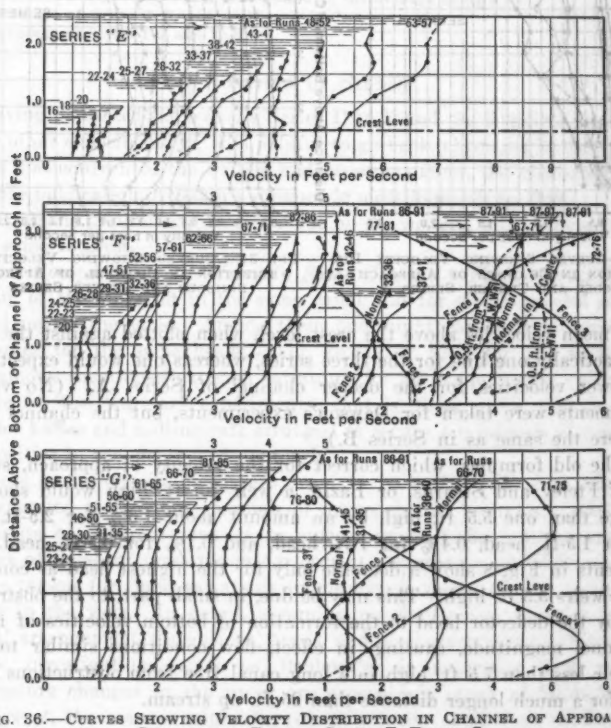

FIG. 36.—CURVES SHOWING VELOCITY DISTRIBUTION IN CHANNEL OF APPROACH, SCHODER AND TURNER, SERIES E, F, AND G.

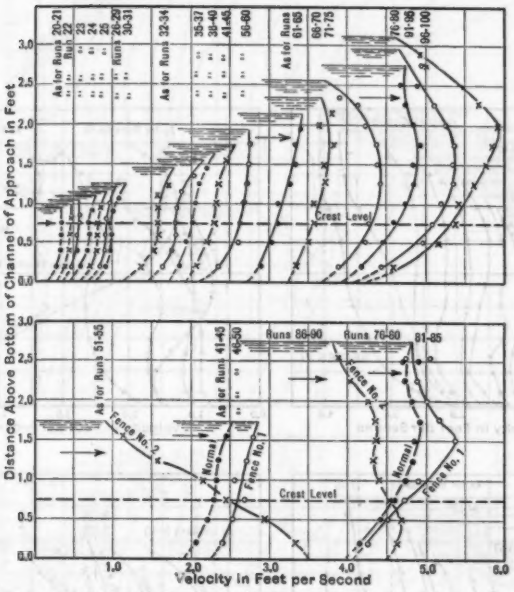

FIG. 37.—CURVES SHOWING VELOCITY DISTRIBUTION IN CHANNEL OF APPROACH, SCHODER AND TURNER, SERIES N.

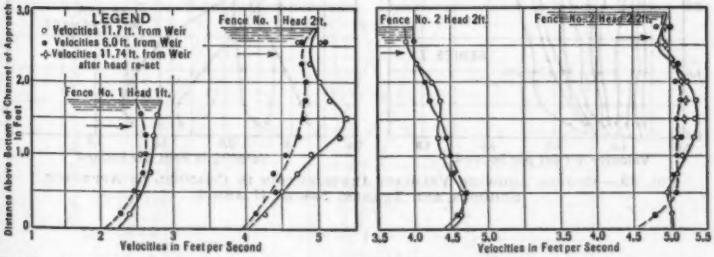

FIG. 38.—CURVES SHOWING COMPARISON OF VELOCITIES AT MID-CHANNEL, AT 11.74 AND 60 FEET UP STREAM FROM WEIR, SCHODER AND TURNER, SERIES N.

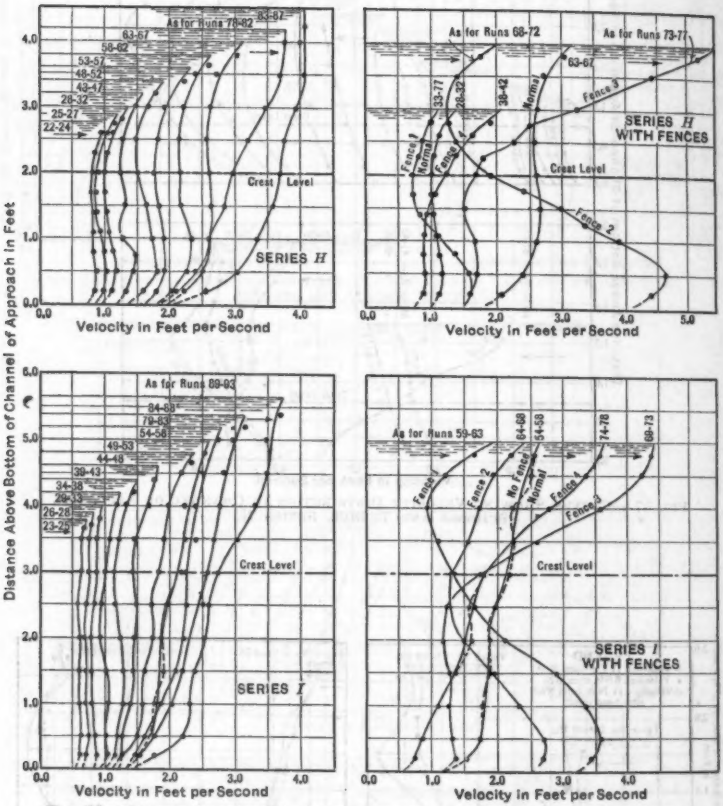

FIG. 39.—CURVES SHOWING VELOCITY DISTRIBUTION IN CHANNEL OF APPROACH. SCHODER AND TURNER, SERIES H AND I.

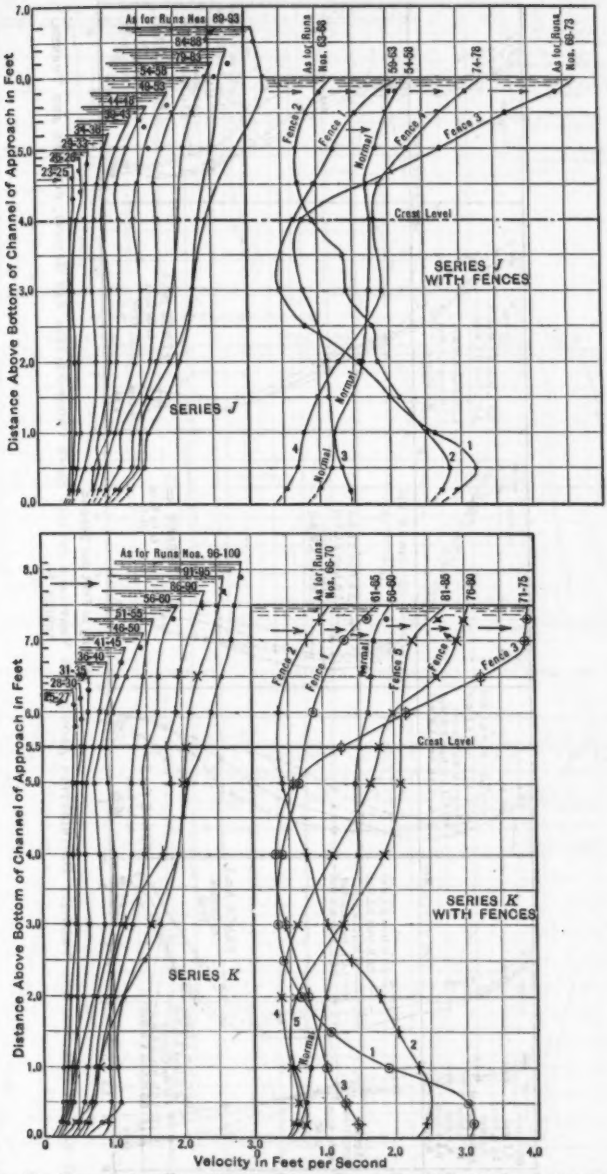

FIG. 40.—CURVES SHOWING VELOCITY DISTRIBUTION IN CHANNELS OF APPROACH, SCHODER AND TURNER, SERIES J AND K.

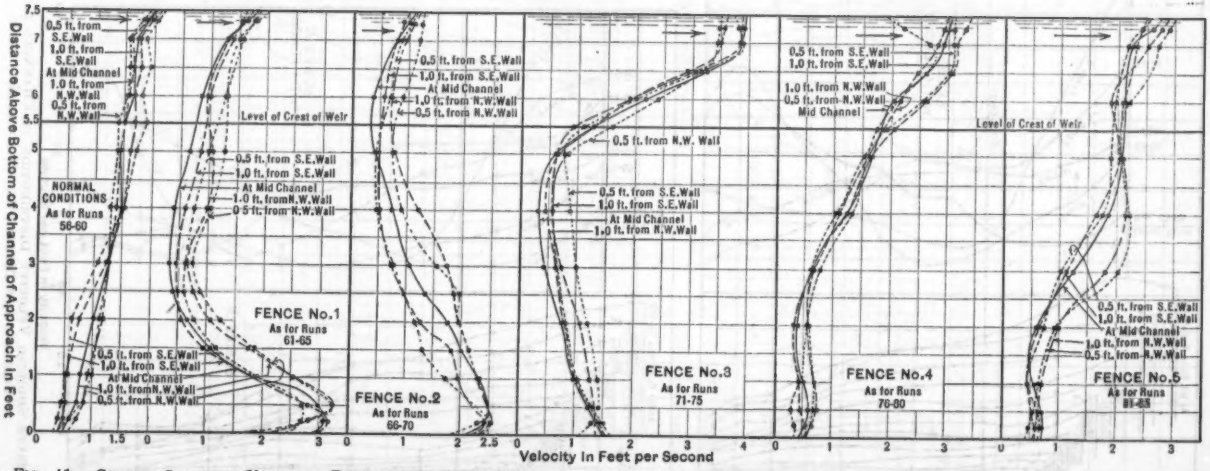

FIG. 41.—CURVES SHOWING VELOCITY DISTRIBUTION ALONG FIVE VERTICALS IN THE SAME CROSS-SECTION (AT GAUGE), SCHODER AND TURNER, SERIES K.

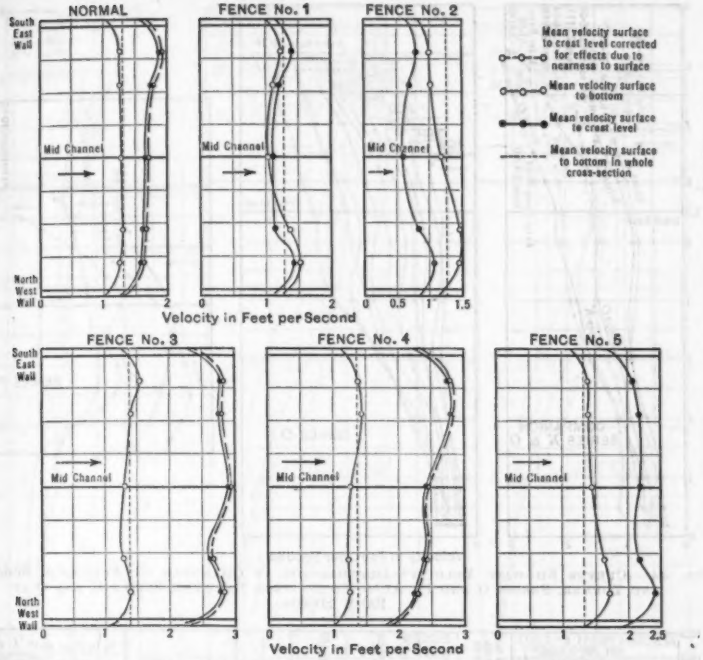

FIG. 42.—CURVES SHOWING VELOCITY DISTRIBUTION HORIZONTALLY, SCHODER AND TURNER, SERIES K.

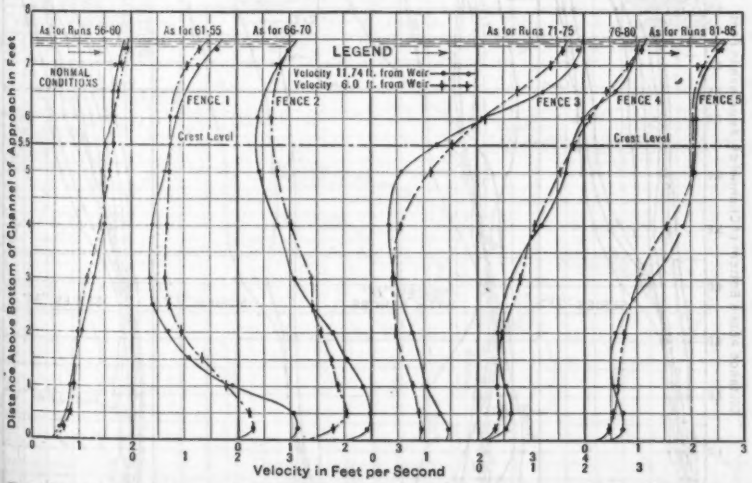

FIG. 43.—CURVES SHOWING COMPARISON OF VELOCITY DISTRIBUTION IN MID-CHANNEL AT 11.74 FEET AND 6.0 FEET UP STREAM FROM WEIR, SCHODER AND TURNER, SERIES K.

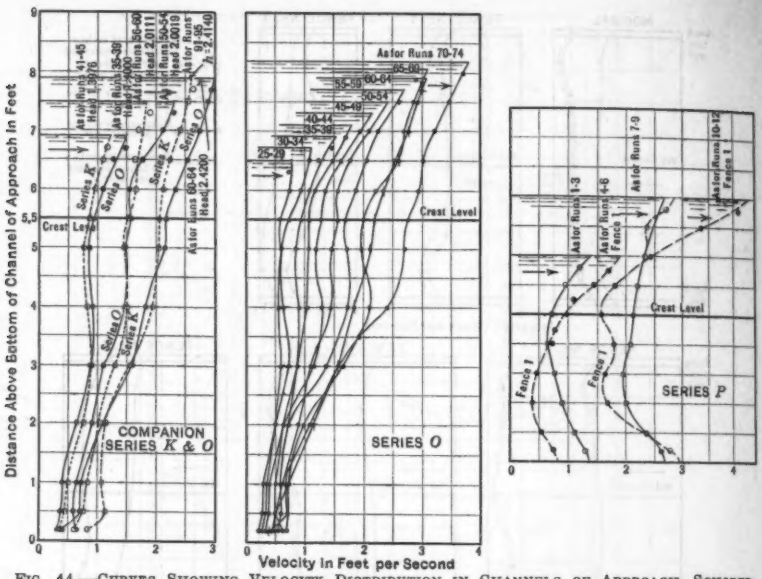

FIG. 44.—CURVES SHOWING VELOCITY DISTRIBUTION IN CHANNELS OF APPROACH, SCHODER AND TURNER, SERIES O AND P; ALSO, COMPARISON BETWEEN SERIES K AND O AT SAME HEADS.

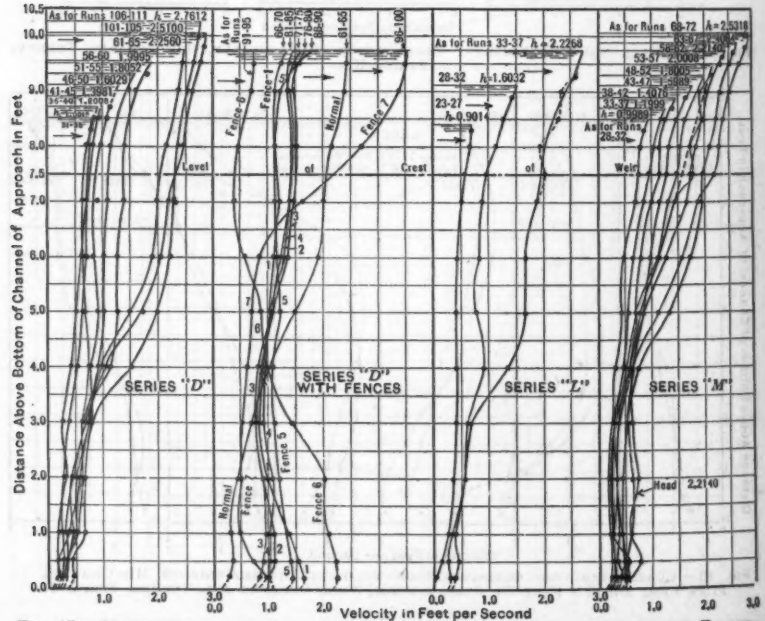

FIG. 45.—CURVES SHOWING COMPARISON OF VELOCITY DISTRIBUTION, SCHODER AND TURNER. SERIES D (NORMAL AND WITH FENCES), L, AND M.

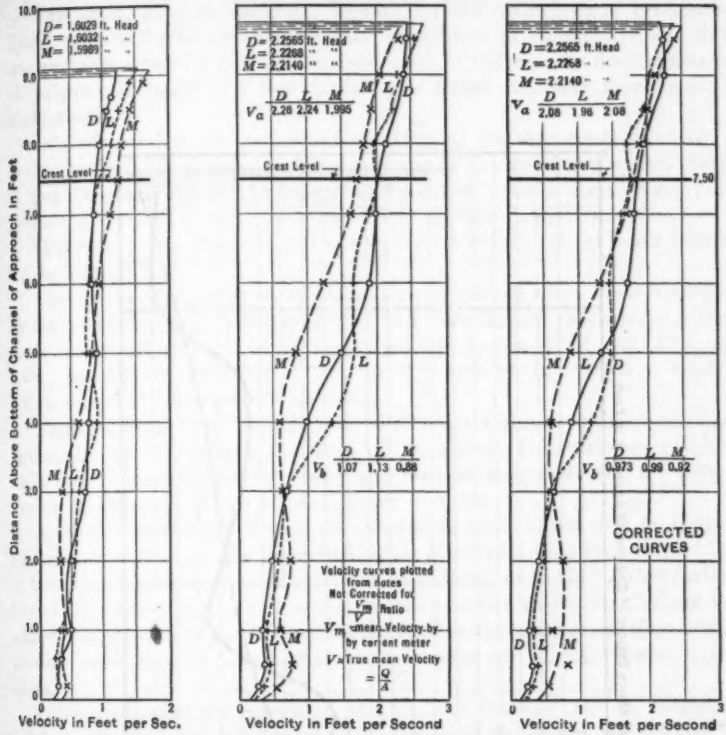

FIG. 46.—CURVES SHOWING COMPARISON OF VELOCITY DISTRIBUTION (MID-CHANNEL) FOR SIMILAR HEADS, SCHODER AND TURNER, SERIES D, L, AND M.

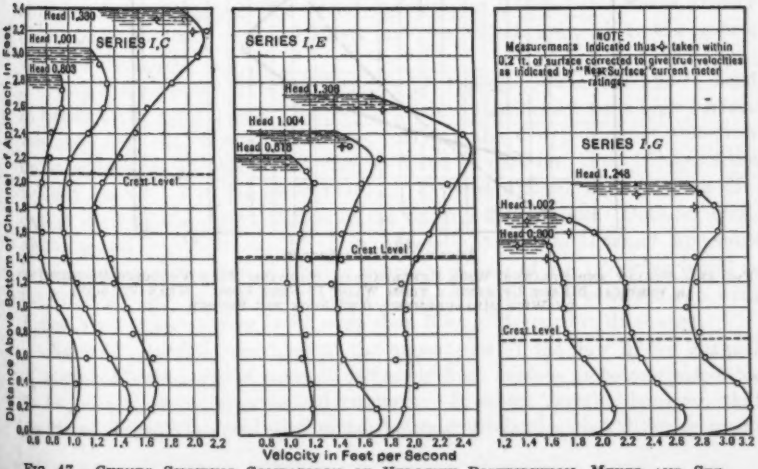

FIG. 47.—CURVES SHOWING COMPARISON OF VELOCITY DISTRIBUTION, MEYER AND SEE, SERIES I-C, I-E, AND I-G.

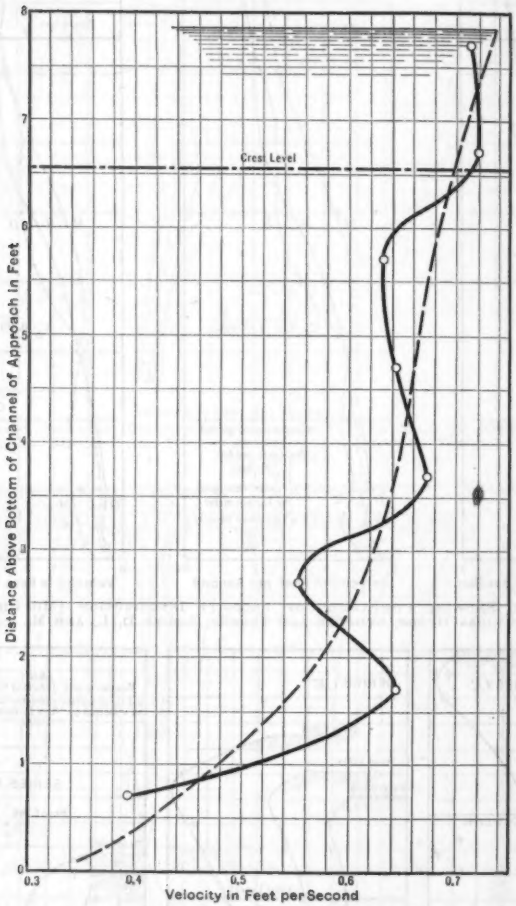

FIG. 48.—FTELEY AND STEARNS' WEIR EXPERIMENTS, SHOWING MEAN VELOCITY DISTRIBUTION IN VERTICAL 5 FEET UP STREAM FROM WEIR, 19 FEET LONG. MEAN VELOCITY BY WEIR MEASUREMENT, 0.63 FOOT PER SECOND.

This effect, however, could not account for the entire lack of conformity. There was a difference in sharpness and smoothness of crest plates, and there was probably a lack of perfect proportionality in the velocity distribution for all heights and heads in a free channel of a length less than four times the highest weir.

(*b*).—Examining the mechanical condition at the weir crests minutely, it is obvious from the descriptions that only two of the weirs, the writers' Series B and Dawson's Series II, deserve to be called strictly sharp-crested; but neither of these two crest pieces had a strictly smooth up-stream face.

It is seen on Fig. 8 that these two series agree within 1% for heads between 0.5 ft. and 2.4 ft.

The considerable effect on discharge due to rusting of an iron crest plate by six weeks' exposure to water in May and June is seen by comparing Dawson's Series II and III. The discharge was increased 1.1, 0.8, 0.75, and 0.6%, respectively, at heads of 0.3, 0.5, 1.0, and 1.5 ft. Above a head of 1.9 ft. the effect seems to be a decrease.

Second Comparison: Old and New Data.—On Fig. 49 are shown the averaging lines for the experimental data (all volumetric measurements on nominally strictly sharp crests) of Fteley and Stearns, Bazin, Martin, the writers' Series B, Dawson's Series II, and Meyer and See.

A cursory inspection of Fig. 49 shows that Bazin's data fall in a group by themselves and indicate distinctly higher discharges than the others. This is true for all heads and heights of weir, as detailed in Table 7. Comparisons have been made with weirs of nearly the same height, except for Column (2), where only the lowest heads are considered. For the other seven comparisons, Bazin's weirs were slightly higher in three cases and slightly lower in four cases.

Other anomalies are apparent. Fteley and Stearns' 1.00-ft. height, at 0.8-ft. head, shows 4.5% excess over the average of Meyer and See's Series 1-F and 2-E of the same height; although at a low head of 0.3 ft., they are in almost exact agreement. The Fteley and Stearns 0.50-ft. weir shows greater discharge at all heads than the Meyer and See 0.41-ft. weir, where the reverse would be expected.

Third Comparison: Effect of Velocity of Approach.—For the 2.25-ft. head in Series C the writers built "fences" to produce in different runs high surface velocities and high bottom velocities, carrying the distribution of velocities up to perhaps the extremes that may be found in practice. Fig. 35 shows the velocity curves. For the same head, an extreme difference of 26% in discharge was produced by variations in the velocity distribution. Data for head, discharge, and velocities are given in Appendix A (Table 11), and Appendix B (Table 55). Even with the lesser re-arrangements of the velocities (such as the "regular" run at the 2.25-ft. head, Series C, compared with the run with Fence No. 3) there is seen to be more than 3.5% difference in discharge.

No doubt careful inspection of the appearance of the flow in the channel of approach would enable cases of extremely high surface or bottom velocities to be noticed by an experienced engineer. It seems certain, however, that no visual inspection could distinguish between discharges that differ as much

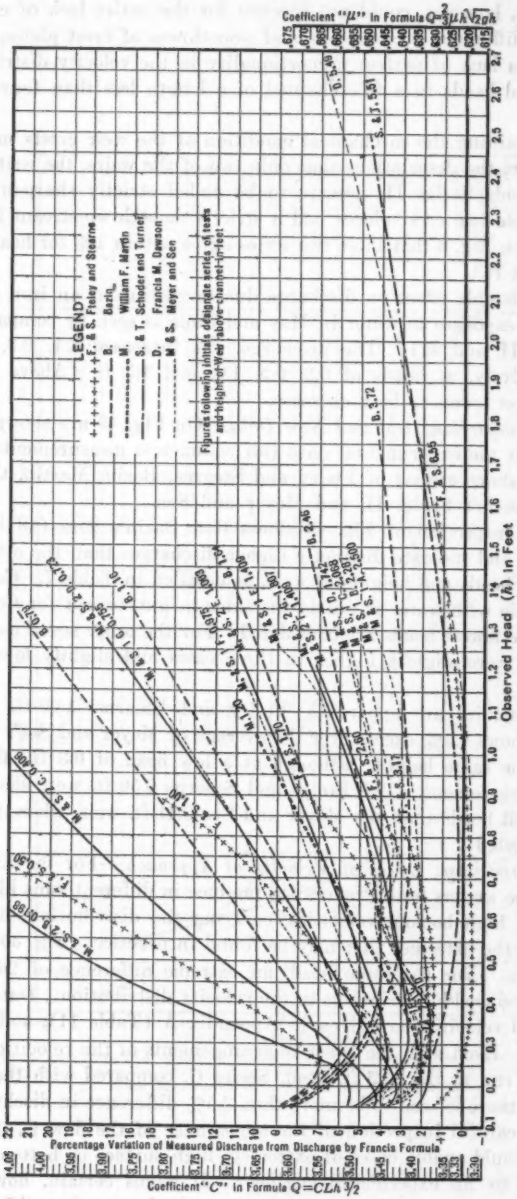

FIG. 49.—SHARP-CRESTED WEIR EXPERIMENTS. COMPARISON OF EARLIER NEW DATA AND THOSE OF FTELEY AND STEARNS AND BAZIN, ALL REFERRED TO FRANCIS FORMULA.

TABLE 8.—COMPARISON OF DISCHARGE AT VARIOUS HEADS FROM EXPERIMENTS BY VARIOUS EXPERIMENTERS ON SHARP AND NEARLY SHARP-CRESTED WEIRS.

(Fifty-seven series of experiments considered in this tabulation. Height of weir varied from 0.198 ft. to 7.53 ft.)

Series.	Height of weir, in feet.	Observed Head on Weir, in Feet.							
		0.20	0.50	0.80	1.00	1.20	1.50	2.00	2.50
		Percentage of excess of discharge over Francis formula.							
S. and T., A	7.53	2.6	0.9	0.7	0.9	1.3	1.9	2.9	3.9
" , D	7.5	1.3	0.0	0.4	0.7	1.7	3.2	3.1	8.4
" , L	7.5	1.8	0.0	0.2	0.6	1.2	2.2	5.0	...
" , M	7.5	1.2	0.4	0.2	0.8	1.5	2.5	4.0	5.6
Fteley and Stearns	6.55	...	—0.5	—0.3	0.1	0.5	1.1	...	...
S. and T., B	5.51	2.3	0.4	0.2	0.4	0.8	1.5	3.0	4.7
Dawson, I	5.51	5.2	2.5	2.0	2.1	2.3	3.0	4.6	6.4
" , ½-in. round	5.51	9.2	3.4	2.3	2.2	2.5	3.1	4.6	6.4
" , II	5.49	0.4	—0.5	—0.4	—0.2	0.2	1.1	3.4	6.0
" , III	5.49	1.7	0.3	0.3	0.5	1.0	1.7	3.1	...
S. and T., C	5.34	3.5	1.1	1.1	1.5	1.9	2.7	4.3	6.1
" , K	5.50	1.0	0.0	—0.1	0.3	0.6	2.2	4.0	6.9
" , O	5.50	0.3	—0.2	0.2	0.7	1.3	2.5	5.1	7.8
" , J	4.00	1.2	—0.4	—0.2	1.0	1.5	2.8	5.4	7.8
Bazin	3.72	6.5	3.1	2.6	2.7	3.0	3.6	...	...
Fteley and Stearns	3.17	2.5	1.4	2.6	...	...	...	...	...
S. and T., I	3.00	1.1	—0.2	0.4	1.6	2.4	3.8	7.0	9.7
Fteley and Stearns	2.60	...	1.8	3.3	...	...	...	...	...
M. and S., I-A	2.50	2.6	1.8	2.2	2.6	3.6	...		
Jones, A. B. H	2.50	1.8	0.8	1.6	2.7	4.2	...		
" , C. D. E	2.50	2.4	1.1	1.9	2.9	4.4	...		
" , F	2.50	3.1	1.6	2.1	3.2	4.6	...		
" , G	2.50	5.0	2.4	2.6	3.4	4.7	...		
Weber, II, III, VI	2.50	1.8	1.0	1.9	3.0	4.5	...		
" , IV	2.50	3.2	2.0	2.6	3.7	5.2	...		
" , V	2.50	3.4	2.5	3.3	4.3	5.6	...		
Bazin	2.46	6.7	3.9	4.6	5.4	6.2	...		
M. and S., I-B	2.287	3.6	2.9	3.2	3.8	4.6	...		
" , I-C	2.068	3.4	3.7	3.9	4.3	5.2	...		
S. and T., H	2.00	0.5	1.0	2.4	3.6	4.8	7.0		
M. and S., 2-G	1.807	3.7	3.1	4.3	5.7	7.3	...		
" , I-D	1.742	2.5	2.9	3.4	4.5	5.9	...		
Fteley and Stearns	1.70	3.0	3.:	5.9	...	...	...		

Series.	Height of weir, in feet.	Observed Head on Weir, in Feet.					
		0.20	0.50	0.80	1.00	1.20	1.50
		Percentage excess over Francis formula.					
Bazin	1.64	6.8	5.0	6.7	8.1	9.6	....
S. and T., G	1.50	1.7	2.0	4.3	6.2	7.9	10.7
M. and S., I-E	1.409	2.1	3.6	4.7	6.5	8.3	....
" 2-F	1.409	1.8	3.1	3.9	5.4	6.9	....
Martin	1.20	2.3	4.0	7.3	9.5	....	....
Bazin	1.16	7.1	6.6	9.6	12.0	14.3	....
M. and S., 2-E	1.003	4.0	4.8	7.2	8.8	10.5	....
Fteley and Stearns	1.00	3.9	6.7	12.0	....	....	....
S. and T., F	1.00	2.0	4.2	8.0	10.5	13.1	17.0
M. and S., I-F	0.975	3.1	5.4	7.8	9.2	10.6	....
Bazin	0.79	7.4	9.1	13.4	16.5	19.5	....
M. and S., I-G	0.755	2.6	7.4	10.5	13.1	15.8	....
S. and T., N	0.75	2.6	5.8	11.0	14.3	17.0	22.0
M. and S., 2-D	0.723	3.2	6.5	10.5	13.3	16.1	....
Fteley and Stearns	0.50	6.4	13.9	....	....	....	....
S. and T., E	0.50	5.2	10.6	18.4	25.0	28.6	37.6
M. and S., 2-C	0.406	4.4	11.6	18.6	22.3	....	....
" , 2-B	0.198	5.5	18.5	27.2	....	....	....

as 4 or 5% at the same head due solely to different distributions of velocities in the channel of approach.

The writers, in view of such conflicts and variations, and needing as reliable an estimate as possible for the discharge over a large reference weir already used by them in other investigations, decided to undertake the studies covered by Series D to P as an auxiliary work.

General Comparison of Old and New Data.—In Table 8 is given a fairly comprehensive comparison of discharges at various heads (0.2-ft. to 2.5-ft.) by various experimenters (old and new) on weirs of various heights, all weirs of the sharp or nearly sharp-crested type. The data are given in terms of the percentage departure from discharges calculated by the simple Francis formula,

$$Q = 3.33\ L\ h^{\frac{3}{2}}, \text{ or } Q = 0.623 \times \frac{2}{3}\ L\ h\ \sqrt{2\,g\,h}$$

All values of the percentage variation are taken from the diagrams, averaging the plotted points, except in the case of the writers' Series D, I, J, K, L, M, N, and O, for which the data are from the actual measurements or interpolated between measurements.

Perhaps this comparison serves best to emphasize the present state of weir hydraulics when refinements of one sort or another are disregarded.

Conclusions Indicated

Supplementary to the main findings given in the Synopsis and to the conclusions stated in the text, or made obvious by tables and diagrams, a few other deductions are of interest.

Condition of Crest Piece.—The data clearly show that both the roughness of the up-stream face and the rounding of the up-stream top corner of the crest plate increase the discharge for the same head, over that for smooth, sharp conditions. A combination of the two effects each in slight degree, could easily amount to from 1 to 2 per cent.

In tests and routine measurements demanding the utmost precision it seems quite necessary to have the crest plate very smooth in order to avoid uncertainties as to the degree of roughness. Whether it is preferable to try to maintain a sharp corner or one definitely rounded to a very small radius may be as much a practical as a hydraulic question. A difficulty with a slightly rounded corner is to have it exactly to dimension and form, but this would not apply to a rounding to, say, $\frac{3}{16}$ or $\frac{1}{4}$ in. radius.

Velocity Distribution in the Channel of Approach.—It seems hardly sufficient to state, as did Bazin, that a certain precision is attainable "provided, however, that the arrangements of our standard weir are exactly reproduced", unless it is indicated clearly which of the arrangements are essential to the "standard weir", which are to be minutely copied, and which are left to the necessities of the particular case or to the judgment of the engineer.

The velocity conditions in the channel of approach should be standardized to some degree for those cases where control is possible. If this were done the use of a formula involving only the head, height, and length of weir

would probably be permissible. A uniform distribution of velocities is not a natural condition for an open channel. Then, too, a measuring weir can seldom be placed conveniently at the down-stream end of a long canal. In practice, the demand is for the shortest possible canal. Often, also, the entry conditions are turbulent, sometimes violently so. The use of baffles seems essential.

In many cases it would be out of the question to depend entirely on the length of the channel of approach to re-arrange the velocities. The measurements of Fteley and Stearns and of the writers indicate that a particular distribution, once started, persists surprisingly in a deep channel. For low weirs with heads higher than the weir, and, consequently, with high velocities of approach, the evidence shows a regularity of a high order as far as discharge is concerned. Figs. 44 and 46 show the differences found by the writers in channels nominally the same.

Measurements of Head.—The side-wall piezometer connected to a stilling-well seems to be an excellent device. However, in addition to the usual precautions concerning perpendicular boring of the hole and avoidance of projections into the channel, the matter of freedom from centrifugal and suction effects due to cross-currents is of some consequence. In recent construction the stilling-well has been connected symmetrically to a pair of opposite side-wall piezometers.

The writers have found the float-gauge, as described, very convenient. A separate setting for each reading is avoided and all the surges can be noted. The use of a glass stilling-pipe allows precise work with a hook-gauge or point-gauge with visibility from below the water surface for the former.

The plumb-bob gauge has the advantages of simplicity, ease of construction, and direct contact with the water surface in the channel of approach. However, it requires a separate setting for each reading.

Friction in the Channel of Approach.—Ordinarily, the effect of friction between the point where the head is measured and the weir is negligibly small; but in an extreme case, as for the writers' Series E (2-ft. head on a weir 0.5 ft. high), the surface fall in 12 ft. for open-channel flow amounts to nearly 1% of the head, and the decrease of discharge for this case is about 1.5% because of friction. For a 1-ft. head the effect on the discharge is still 1 per cent.

For very narrow channels, Martin's experiments indicate that allowances for friction, by open channel laws, between the head gauge station and the weir, are adequate subtractive corrections for the head. The writers' large-scale experiments furnish more evidence on this point.

Acknowledgments

It is not intended to imply that there may not long have been available to a few engineers and scientists working information on the points mentioned in the paper. The writers know that all the points specified in the title have been the subjects of not a little consideration and some experimentation previously. To some extent due acknowledgment is made in the paper, but it is undoubtedly inadequate and incomplete. The writers would welcome additional prior or contemporaneous evidence.

The completion of this paper has been greatly assisted by grants from the Heckscher Research Foundation of Cornell University in 1922, 1923, and 1925. Professor Francis M. Dawson was Research Assistant under these grants. The great aid rendered by him in the preparation of the paper is appreciatively and gratefully acknowledged.

(Professor Schoder wishes to record his profound regret and sense of a great loss to engineering science in that Professor Turner's life was cut short before the completion of these studies for publication; and yet it was a great privilege to enjoy the long scientific comradeship and deep friendship that existed between the writers as partners during the strenuous experimentation involved in these and the other (still unpublished) researches.)

APPENDIX A

TABLE 9.—SCHODER AND TURNER, (NEARLY) SHARP-CRESTED WEIRS, SERIES A.

No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis formula.
	Float-gauge.	Plumb-bob gauge.				
1	0.0494	0.0502	0.1767	0.0498	0.0419	13 2
2	0.0604	0.0612	0.2227	0.0608	0.0544	9.0
3	0.0774	0.0793	0.3351	0.0783	0.0794	8.9
4	0.0904	0.0910	0.4104	0.0907	0.0973	6.9
5	0.1148	0.1162	0.5837	0.1155	0.1383	5.7
6	0.1546	0.1557	0.8940	0.1552	0.2118	4.0
7	0.1788	0.1802	1.1090	0.1795	0.2607	3.0
8	0.1978	0.1980	1.3674	0.1979	0.3003	2.4
9	0.2529	0.2553	1.8367	0.2541	0.4352	2.0
10	0.2948	0.2954	2.1809	0.2951	0.5166	1.9
11	0.3306	0.3212	2.5913	0.3209	0.6141	1.5
12	0.3717	0.3721	3.2277	0.3719	0.7649	1.3
13	0.3959	0.3963	3.5491	0.3961	0.8410	1.3
14	0.4209	0.4211	3.8877	0.4210	0.9213	1.3
15	0.4672	0.4682	4.5463	0.4677	1.0773	1.1
16	0.4972	0.4984	4.9739	0.4978	1.1787	0.8
17	0.5486	0.5501	5.7340	0.5502	1.3641	0.4
18	0.5499	0.5520	5.7790			
19	0.7590	0.7612	9.3633	0.7602	2.2216	0.7
20	0.7598	0.7610	9.3996			
21	0.7597	0.7610	9.4155			
22	0.7596	0.7598	9.3993			
23	0.7599	0.7608	9.4082			
24	0.8788	0.8800	11.670	0.8793	2.7645	0.7
25	0.8788	0.8800	11.718			
26	0.8790	0.8800	11.716			
27	0.8786	0.8793	11.743	0.9989	3.3607	1.1
28	0.8788	0.8800	11.633			
29	0.9986	0.9993	14.177			
30	0.9988	0.9991	14.227			
31	0.9982	0.9992	14.212			
32	0.9992	0.9990	14.232			
33	0.9982	0.9987	14.208			
34	1.1954	1.1968	18.597	1.1968	4.4249	1.5
35	1.1954	1.1964	18.743			
36	1.1963	1.1968	18.715			
37	1.1968	1.1981	18.711			
38	1.1965	1.1993	18.765			
39	1.4114	1.4111	23.885	1.4115	5.6642	1.4
40	1.4106	1.4116	23.953			
41	1.4110	1.4121	23.974			
42	1.4121	1.4120	23.980			
43	1.7043	1.7063	32.099	1.7088	7.6223	2.5
44	1.7082	1.7096	32.254			
45	1.7098	1.7108	32.243			
46	1.7100	1.7108	32.275			
47	1.7094	1.7085	32.248			
48	1.9090	1.9117	38.428	1.9133	9.1237	3.5
49	1.9124	1.9147	38.530			
50	1.9124	1.9147	38.626			
51	1.9144	1.9148	38.703			
52	1.9144	1.9148	38.647			
53	2.1612	.....	45.908	2.1572	10.885	3.2
54	2.1532	.....	46.080			
55	2.1612	.....	46.247			
56	2.1532	.....	46.108			
57	2.1572	.....	46.076			
58	2.2767	.....	49.900	2.2767	11.784	3.0
59	2.4332	.....	55.911	2.4398	13.150	3.6
60	2.4502	.....	56.253			
61	2.4437	.....	55.440			
62	2.4327	.....	55.593			
63	2.4392	.....	55.436			

TABLE 10.—SCHODER AND TURNER, SHARP-CRESTED WEIRS, SERIES B.

No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.	No. of run.	Observed Head, in Feet		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.
	Float-gauge.	Plumb-bob gauge.						Float-gauge.	Plumb-bob gauge.				
1	0.0496	0.0510	0.1778	0.0504	0.0414	10.6	39	1.4088	.....	23.904			
2	0.0499	0.0509	0.1743				40	1.4218	.....	24.164			
3	0.0636	0.0639	0.2567	0.0638	0.0608	13.2	41	1.4210	.....	24.171	1.4174	5.7085	1.6
4	0.0774	0.0780	0.3621	0.0777	0.0858	19.0	42	1.4176	.....	24.134			
5	0.0969	0.0986	0.4693	0.0978	0.1112	9.1	43	1.4193	.....	24.115			
6	0.1169	0.1179	0.6020	0.1174	0.1427	6.6	44	1.4160	.....	24.050			
7	0.1379	0.1393	0.7613	0.1386	0.1804	5.0	45	1.7061	.....	31.783			
8	0.1648	0.1659	0.9777	0.1654	0.2317	3.4	46	1.7084	.....	31.918			
9	0.1830	0.1845	1.1358	0.1838	0.2691	2.6	47	1.7096	.....	31.840	1.7088	7.5590	1.6
10	0.2032	0.2039	1.3193	0.2036	0.3126	2.2	48	1.7098	.....	32.024			
11	0.2488	0.2499	1.7813	0.2494	0.4221	1.8	49	1.7101	.....	31.980			
12	0.3018	0.3019	2.3542	0.3018	0.5579	1.1	50	1.9128	.....	37.975			
13	0.3526	0.3524	2.9730	0.3525	0.7045	1.1	51	1.9035	.....	37.848			
14	0.3983	0.3994	3.5537	0.3988	0.8421	0.4	52	1.9098	.....	38.142	1.9092	9.0018	2.5
15	0.4520	0.4525	4.2910	0.4522	1.0168	0.4	53	1.9081	.....	37.917			
16	0.4923	0.4925	4.8860	0.4952	1.1683	0.7	54	1.9118	.....	38.055			
17	0.4979	0.4981	4.9744				55	2.1480	.....	45.777			
18	0.5519	0.5527	5.7796	0.5523	1.3696	0.2	56	2.1536	.....	45.914			
19	0.7536	0.7549	9.1935				57	2.1530	.....	45.861	2.1513	10.865	3.4
20	0.7522	0.7537	9.1630				58	2.1520	.....	45.834			
21	0.7516	0.7537	9.1428	0.7528	2.1698	− 0.02	59	2.1498	.....	45.871			
22	0.7517	0.7537	9.2145				60	2.2758	.....	50.256			
23	0.7509	0.7519	9.0698				61	2.2673	.....	50.237			
24	0.8700	.....	11.449				62	2.2688	.....	49.958	2.2698	11.897	4.5
25	0.8720	.....	11.499				63	2.2725	.....	50.360			
26	0.8721	.....	11.539	0.8719	2.7291	0.7	64	2.2648	.....	50.226			
27	0.8732	.....	11.555				65	2.4346	.....	55.783			
28	0.8724	.....	11.541				66	2.4426	.....	55.737			
29	1.0011	.....	14.137				67	2.4336	.....	56.005	2.4393	13.256	4.5
30	1.0015	.....	14.185				68	2.4359	.....	55.721			
31	1.0002	.....	14.193	1.0020	3.3655	0.8	69	2.4496	.....	56.462			
32	1.0036	.....	14.232										
33	1.0037	.....	14.265										
34	1.2076	.....	18.727										
35	1.2068	.....	18.743										
36	1.2066	.....	18.784	1.2079	4.4519	0.7							
37	1.2064	.....	18.781										
38	1.2120	.....	18.898										

TABLE 11.—SCHODER AND TURNER, (NEARLY) SHARP-CRESTED WEIRS, SERIES C.

No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis formula.
	Float-gauge.	Plumb-bob gauge.				
1	0.0363	0.0366	0.0850	0.0364	0.0201	—13.0
2	0.0384	0.0384	0.0990	0.0384	0.0235	— 6.4
3	0.0418	0.0420	0.1175	0.0419	0.0278	— 2.8
4	0.0443	0.0447	0.1302	0.0445	0.0309	— 1.0
* 5	0.0497	0.0487	0.1603	0.0489	0.0383	+ 6.4
* 6	0.0482	0.0489	0.1630			
7	0.0536	0.0538	0.1920	0.0537	0.0455	9.9
* 8	0.0553	0.0549	0.1990	0.0551	0.0472	9.5
* 9	0.0612	0.0615	0.2407	0.0612	0.0571	13.3
* 10	0.0611	0.0612	0.2413			
* 11	0.0723	0.0719	0.3212	0.0721	0.0763	18.5
* 12	0.0720	0.0722	0.3230			
13	0.0810	0.0816	0.3800	0.0813	0.0900	16.6
* 14	0.0815	0.0831	0.3987	0.0827	0.0945	19.3
* 15	0.0829	0.0833	0.3993			
16	0.0837	0.0832	0.4077	0.0834	0.0966	20.4
17	0.0842	0.0837	0.4157	0.0840	0.0985	21.5
18	0.0857	0.0849	0.4113	0.0853	0.0975	17.5
19	0.0877	0.0878	0.4213	0.0878	0.1005	16.1
20	0.0879	0.0879	0.4197	0.0879	0.0995	14.6
21	0.0885	0.0878	0.4383	0.0882	0.1039	19.2
* 22	0.0904	0.0894	0.4573	0.0902	0.1088	20.6
* 23	0.0910	0.0899	0.4610			
24	0.0910	0.0914	0.4390	0.0912	0.1040	13.4
25	0.0920	0.0919	0.4740	0.0920	0.1123	20.9
26	0.0926	0.0928	0.4593	0.0927	0.1088	15.7
27	0.0933	0.0935	0.4483	0.0934	0.1062	11.8
28	0.0936	0.0937	0.4777	0.0936	0.1132	18.7
29	0.0943	0.0935	0 4700	0.0939	0.1114	16.3
30	0.0964	0.0962	0.5096	0.0963	0.1208	21.4
* 31	0.1013	0.1004	0.4993	0.1014	0.1183	10.0
* 32	0.1029	0.1010	0.4990			
33	0.1018	0.1022	0.4947	0.1020	0.1172	8.0
34	0.1025	0.1024	0.5077	0.1024	0.1203	10.3
35	0.1082	0.1071	0.5413	0.1076	0.1283	9.1
36	0.1087	0.1079	0.5367	0.1083	0.1272	7.2
37	0.1092	0.1096	0.5530	0.1094	0.1310	8.8
38	0.1117	0.1114	0.5692	0.1116	0.1349	8.7
39	0.1156	0.1157	0.5870	0.1156	0.1391	6.3
* 40	0.1194	0.1194	0.6217	0.1196	0.1477	7.3
* 41	0.1196	0.1198	0.6447			
* 42	0.1382	0.1370	0.7573	0.1378	0.1794	5.3
* 43	0.1391	0.1389	0.7570			
44	0.1440	0.1436	0.7900	0.1438	0.1872	3.1
45	0.1616	0.1605	0.9483	0.1609	0.2249	4.7
46	0.1606	0.1608	0.9500			
47	0.1800	0.1797	1.1163	0.1801	0.2646	4.0
48	0.1801	0.1805	1.1173			
49	0.2006	0.1997	1.3029	0.2002	0.3091	3.6
50	0.2004	0.2008	1.3058			
51	0.2514	0.2507	1.8139	0.2510	0.4293	2.5
52	0.2518	0.2508	1.8094			
53	0.2996	0.2985	2.3434	0.2988	0.5567	2.4
54	0.2990	0.2982	2.3555			
55	0.3501	0.3503	2.9547	0.3501	0.7010	1.6
56	0.3502	0.3499	2.9617			
57	0.3996	0.3991	3.5900	0.3992	0.8518	1.4
58	0.3994	0.3990	3.5990			
59	0.4502	0.4496	4.2850	0.4500	1.0177	1.2
60	0.4501	0.4500	4.3040			
61	0.5024	0.5023	5.0571	0.5026	1.2000	1.1
62	0.5029	0.5027	5.0708			
63	0.5995	0.5985	6.5624	0.5987	1.5585	1.0
64	0.5990	0.5977	6.5914			
65	1.0000	.....	14.270	1.0006	3.3831	1.5
66	1.0004	.....	14.285			
67	1.0014	.....	14.275			
68	1.2062	.....	18.953	1.2058	4.5005	2.1
69	1.2054	.....	19.031			
70	1.2048	.....	18.993			
71	1.4034	.....	21.899	1.3994	5.6361	2.2
72	1.4014	.....	23.843			
73	1.3986	.....	23.820			
74	1.3958	.....	23.624			
75	1.3976	.....	23.735	1.5897	6.8664	2.9
76	1.5964	.....	29.118			
77	1.5900	.....	28.956			
78	1.5884	.....	28.964			
79	1.5863	.....	28.866			
80	1.8034	.....	35.022	1.7947	8.2805	3.4
81	1.7939	.....	34.944			
82	1.7933	.....	34.951			
83	1.7906	.....	34.914			
84	1.7921	.....	34.887			
85	2.0030	.....	41.622	1.9983	9.8224	4.4
86	1.9996	.....	41.467			
87	1.9996	.....	41.462			
88	1.9966	.....	41.374			
89	1.9928	.....	41.327			
90	2.1976	.....	48.510	2.1937	11.427	5.6
91	2.1941	.....	48.317			
92	2.1956	.....	48.264			
93	2.1894	.....	47.970			
94	2.1918	.....	48.038			
95	2.2475	.....	49.778	2.2442	11.750	5.0
96	2.2446	.....	49.720			
97	2.2439	.....	49.535			
98	2.2411	.....	49.308			
99	2.2438	.....	49.595			
100	2.2115	.....	59.996	2.2427	14.211 (Fence 1)	27.1
101	2.2434	.....	60.072			
102	2.2447	.....	60.332			
103	2.2405	.....	59.800			
104	2.2439	.....	59.646			
105	2.2614	.....	55.652	2.2711	13.312 (Fence 2)	16.8
106	2.2591	.....	55.866			
107	2.2604	.....	55.862			
108	2.2376	.....	56.682			
109	2.2371	.....	56.811			
110	2.2661	.....	52.094	2.2630	12.315 (Fence 3)	8.6
111	2.2611	.....	51.632			
112	2.2614	.....	51.943			
113	2.2644	.....	52.161			
114	2.2619	.....	52.056			
115	2.2679	.....	48.506	2.2670	11.490 (Fence 4)	1.1
116	2.2651	.....	48.311			
117	2.2674	.....	48.508			
118	2.2668	.....	48.630			
119	2.2676	.....	48.483			
120	2.2737	.....	54.175	2.3729	12.823	5.3
121	2.3776	.....	54.180			
122	2.3751	.....	54.157			
123	2.3684	.....	54.155			
124	2.3698	.....	53.899			
125	2.5312	.....	60.011	2.5304	14.237	6.2
126	2.5282	.....	60.089			
127	2.5318	.....	60.144			

* Runs made with old type float.

TABLE 12.—SCHODER AND TURNER, SHARP-CRESTED WEIRS, SERIES D.

No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.
	Float-gauge.	Plumb-bob gauge.				
1	0.0284	0.0279	0.0740	0.0282	0.0175	10.8
2	0.0315	0.0317	0.0910	0.0316	0.0216	15.5
3	0.0350	0.0349	0.1040	0.0350	0.0246	12.8
4	0.0395	0.0391	0.1197	0.0393	0.0284	9.7
5	0.0459	0.0459	0.1515	0.0459	0.0359	9.5
6	0.0525	0.0523	0.1830	0.0524	0.0434	8.8
7	0.0601	0.0605	0.2207	0.0603	0.0523	6.1
8	0.0677	0.0673	0.2623	0.0675	0.0622	6.5
9	0.0771	0.0769	0.3163	0.0770	0.0750	5.3
10	0.0873	0.0867	0.3762	0.0870	0.0891	4.2
11	0.0987	0.0979	0.4515	0.0983	0.1070	4.3
12	0.1145	0.1145	0.5617	0.1145	0.1331	3.2
13	0.1349	0.1345	0.7110	0.1347	0.1685	2.4
14	0.1559	0.1555	0.8782	0.1557	0.2081	1.7
15	0.1802	0.1807	1 0922	0.1804	0.2588	1.5
16	0.2008	0.2005	1.2773	0.2004	0.3027	1.3
17	0.2501	0.2493	1.7683	0.2497	0.4190	0.8
18	0.3019	0.3015	2.3467	0.3017	0.5561	0.8
19	0.3511	0.3510	2.9413	0.3510	0.6970	0.6
20	0.4005	0.4009	3.5747	0.4007	0.8471	0.3
21	0.4539	0.4533	4.3083	0.4536	1.0209	0.4
22	0.5035	0.5029	5.0138	0.5031	1.1885	0.0
23	0.5033	0.5028	5.0175			
24	0.8061	0.8073	10.258			
25	0.8061	0.8059	10.184	0.8066	2.4218	0.4
26	0.8060	0.8069	10.219			
27	0.8072	0.8073	10.221			
28	0.9001	0.8986	12.052			
29	0.8993	0.8989	12.058	0.8990	2.8559	0.6
30	0.8988	0.8986	12.045			
31	1.0036	1.0037	14.216			
32	1.0035	1.0046	14.214			
33	1.0029	1.0041	14.240	1.0039	3.3732	0.7
34	1.0031	1.0051	14.256			
35	1.0035	1.0045	14.249			
36	1.2046	1.2068	18.862			
37	1.2061	1.2071	18.885			
38	1.2063	1.2059	18.974	1.2066	4.4865	1.7
39	1.2081	1.2103	18.962			
40	1.2072	1.2089	18.984			
41	1.3969	1.3966	23.803			
42	1.3954	1.3951	23.852			
43	1.3964	1.3971	23.864	1.3967	5.6479	2.8
44	1.3972	1.3961	23.858			
45	1.3974	1.3986	23.794			
46	1.5951	1.5928	29.322			
47	1.5952	1.5954	29.359			
48	1.5932	1.5964	29.277	1.5945	6.9455	3.6
49	1.5961	1.5954	29.350			
50	1.5939	1.5914	29.243			
51	1.8047	1.8073	34.883			
52	1.7992	1.8055	34.865			
53	1.8020	1.8072	34.854	1.8046	8.2784	2.5
54	1.8058	1.8050	35.088			
55	1.8050	1.8048	34.986			
56	2.0030	2.0016	40.889			
57	1.9985	2.0018	40.989			
58	2.0021	1.9998	41.047	2.0015	9.7185	3.1
59	2.0020	2.0007	41.161			
60	2.0034	2.0017	40.975			
61	2.2678	2.2734	50.974			
62	2.2829	2.2792	51.205			
63	2.2451	2.2396	50.284	2.2582	12.002	6.2
64	2.2391	2.2381	50.138			
* 65	2.2449	2.2469	48.965			
66	2.2498	2.2540	47.723			
67	2.2497	2.2564	47.967			
68	2.2556	2.2554	48.122	2.2589	11.372 (Fence 1)	0.9
69	2.2555	2.2563	48.146			
70	2.2518	2.2544	47.983			
71	2.2479	2.2520	48.050			
72	2.2478	2.2502	47.994			
73	2.2471	2.2478	48.174	2.2474	11.402 (Fence 2)	1.6
74	2.2462	2.2462	48.041			
75	2.2469	2.2414	48.317			
76	2.2515	2.2522	48.791			
77	2.2512	2.2531	48.614			
78	2.2535	2.2508	48.685	2.2528	11.551 (Fence 3)	2.6
79	2.2552	2.2508	48.764			
80	2.2536	2.2556	48.872			
81	2.2520	2.2533	48.481			
82	2.2484	2.2487	48.397			
83	2.2471	2.2488	48.340	2.2501	11.472 (Fence 4)	2.1
84	2.2526	2.2516	48.402			
85	2.2480	2.2506	48.438			
86	2.2473	2.2510	47.984			
87	2.2456	2.2523	48.009			
88	2.2465	2.2492	47.977	2.2484	11 373 (Fence 5)	+1.3
89	2.2455	2.2503	47.895			
90	2.2465	2.2502	48.095			
91	2.2476	2.2554	47.173			
92	2.2492	2.2546	47.063			
93	2.2448	2.2570	47.099	2.2503	11.151 (Fence 6)	−0.8
94	2.2464	2.2554	47.037			
95	2.2431	2.2555	46.922			
96	2.2535	2.2561	51.094			
97	2.2518	2.2456	51.281			
98	2.2499	2.2499	50.730	2.2510	12.071 (Fence 7)	+7.3
99	2.2520	2.2521	50.685			
100	2.2468	2.2519	50.912			
101	2.5062	2.5082	60.796			
102	2.5106	2.5077	60.432			
103	2.5045	2.5037	60.106	2.5057	14.320	8.4
104	2.5012	2.5014	60.452			
105	2.5092	2.5044	60.357			
106	2.7412	2.7399	68.648			
107	2.7288	2.7231	67.324			
108	2.7602	2.7581	69.270	2.7490	16.234	7.0
109	2.7599	2.7555	69.181			
110	2.7569	2.7536	68.502			
111	2.7598	2.7515	68.181			

* This run not used in averages.

TABLE 13.—SCHODER AND TURNER, SHARP-CRESTED WEIRS, SERIES E.

No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.	No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.
	Float-gauge.	Plumb-bob gauge.						Float-gauge.	Plumb-bob gauge.				
1	0.0288	0.0287	0.0765	0.0288	0.0181	11.7	33	1.2072	.....	24.126			
2	0.0350	0.0334	0.1043	0.0342	0.0247	17.1	34	1.2095	.....	24.198			
3	0.0382	0.0373	0.1160	0.0378	0.0275	12.2	35	1.2055	.....	24.092	1.2090	5.7344	29.5
4	0.0424	0.0418	0.1357	0.0421	0.0322	11.8	36	1.2112	.....	24.240			
5	0.0514	0.0508	0.1763	0.0511	0.0418	8.6	37	1.2114	.....	24.339			
6	0.0608	0.0606	0.2285	0.0607	0.0541	8.6	38	1.4093	.....	31.836			
7	0.0733	0.0727	0.2978	0.0730	0.0706	7.5	39	1.4145	.....	31.968			
8	0.0814	0.0809	0.3470	0.0812	0.0822	6.6	40	1.4120	.....	31.971	1.4128	7.5775	35.5
9	0.0820	0.0820	0.3495	0.0820	0.0828	5.9	41	1.4131	.....	32.219			
10	0.0921	0.0920	0.4062	0.0920	0.0967	4.1	42	1.4152	.....	31.953			
11	0.1044	0.1038	0.4897	0.1041	0.1160	3.7	43	1.5991	.....	39.684			
12	0.1251	0.1239	0.6525	0.1245	0.1546	5.7	44	1.6032	.....	39.722			
13	0.1457	0.1454	0.8198	0.1456	0.1942	5.0	45	1.6025	.....	39.892	1.5991	9.4149	39.8
14	0.1671	0.1663	1.0018	0.1667	0.2374	4.7	46	1.5985	.....	39.840			
15	0.1876	0.1864	1.1933	0.1870	0.2828	5.0	47	1.5920	.....	39.515			
16	0.2078	0.2066	1.3970	0.2072	0.3310	5.4	48	1.7933	.....	48.966			
17	0.2539	0.2524	1.8940	0.2532	0.4488	5.8	49	1.7978	.....	49.220			
18	0.3031	0.3019	2.4868	0.3025	0.5898	6.4	50	1.7980	.....	49.354	1.7970	11.649	45.2
19	0.3494	0.3478	3.1138	0.3486	0.7379	7.7	51	1.7980	.....	49.245			
20	0.4021	0.4014	3.8803	0.4018	0.9195	7.8	52	1.7980	.....	49.012			
21	0.4517	0.4515	4.6650	0.4516	1.1055	9.4	53	1.9955	.....	60.813			
22	0.8000	0.8031	11.875				54	2.0026	.....	60.223			
23	0.7994	0.8043	11.894	0.8016	2.8192	18.0	55	2.0016	.....	61.057	2.0001	14.434	53.2
24	0.8000	0.8026	11.900				56	2.0037	.....	61.383			
25	0.8986	.....	14.622				57	1.9972	.....	61.078			
26	0.8971	.....	14.634	0.8979	3.4635	22.2							
27	0.8979	.....	14.608										
28	1.0043	.....	17.665										
29	1.0043	.....	17.676										
30	1.0039	.....	17.661										
31	1.0056	.....	17.663	1.0044	4.1858	24.9							
32	1.0038	.....	17.654										

TABLE 14.—SCHODER AND TURNER, SHARP-CRESTED WEIRS, SERIES F.

No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.	No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.
	Float-gauge.	Plumb-bob gauge.						Float-gauge.	Plumb-bob gauge.				
1	0.0289	0.0285	0.0752	0.0287	0.0178	9.9	47	1.2036	.....	21.014			
2	0.0337	0.0342	0.1008	0.0340	0.0239	14.4	48	1.2043	.....	21.005			
3	0.0354	0.0351	0.1063	0.0352	0.0252	14.5	49	1.2023	.....	20.959	1.2035	4.9718	13.1
4	0.0421	0.0421	0.1327	0.0422	0.0314	8.7	50	1.2030	.....	20.927			
5	0.0490	0.0484	0.1632	0.0487	0.0387	8.1	51	1.2043	.....	20.998			
6	0.0571	0.0567	0.2038	0.0569	0.0483	6.9	52	1.3988	.....	26.332			
7	0.0638	0.0636	0.2415	0.0637	0.0572	6.9	53	1.3994	.....	26.384			
8	0.0738	0.0735	0.2978	0.0736	0.0706	6.2	54	1.3974	.....	26.891	1.3977	6.3701	15.8
9	0.0848	0.0842	0.3628	0.0845	0.0860	5.1	55	1.3995	.....	26.888			
10	0.0949	0.0947	0.4295	0.0948	0.1018	4.7	56	1.3983	.....	26.814			
11	0.1069	0.1067	0.5057	0.1068	0.1198	3.0	57	1.6008	.....	33.751			
12	0.1206	0.1205	0.6000	0.1206	0.1422	1.9	58	1.6016	.....	33.644			
13	0.1416	0.1412	0.7683	0.1414	0.1821	2.8	59	1.6034	.....	33.692	1.6012	7.9761	18.2
14	0.1598	0.1595	0.9137	0.1596	0.2165	2.0	60	1.5969	.....	33.578			
15	0.1802	0.1796	1.0943	0.1799	0.2593	2.0	61	1.6034	.....	33.630			
16	0.2001	0.1996	1.2803	0.1998	0.3034	2.0	62	1.8050	.....	41.271			
17	0.2509	0.2503	1.8062	0.2506	0.4280	2.5	63	1.8028	.....	41.307			
18	0.2996	0.2978	2.2397	0.2987	0.5544	2.0	64	1.8055	.....	41.334	1.8021	9.8010	21.7
19	0.3529	0.3522	3.0310	0.3526	0.7182	3.0	65	1.7991	.....	41.494			
20	0.4000	0.3997	3.6570	0.3998	0.8666	2.9	66	1.7982	.....	41.392			
21	0.4508	0.4507	4.4080	0.4508	1.0446	3.6	67	1.9992	.....	48.962			
22	0.5012	0.4997	5.2037	0.5008	1.2299	4.2	68	2.0045	.....	49.561			
23	0.5014	0.5009	5.1763				69	1.9983	.....	49.069	2.0008	11.659	23.7
24	0.6991	0.7018	8.8017	0.7010	2.0858	6.7	70	2.0065	.....	49.432			
25	0.7012	0.7018	8.8025				71	1.9957	.....	48.980			
26	0.7990	0.7994	10.840				72	2.0018	.....	51.410			
27	0.7983	0.7994	10.824	0.7990	2.5675	8.0	73	2.0009	.....	51.468			
28	0.7986	0.7992	10.841				74	2.0098	.....	51.403	2.0024	12.219	29.5
29	0.9028	0.9028	13.125				75	1.9944	.....	51.886		(Fence 1)	
30	0.9043	0.9026	13.175	0.9027	3.1182	9.2	76	2.0050	.....	51.652			
31	0.9019	0.9021	13.178				77	1.9975	.....	50.597			
32	0.9996	.....	15.470				78	1.9869	.....	50.070			
33	1.0033	.....	15.628				79	1.9973	.....	50.389	1.9928	11.931	27.4
34	1.0036	.... .	15.625	1.0028	3.6964	10.5	80	1.9923	.....	50.211		(Fence 3)	
35	1.0043	.....	15.635				81	1.9902	.....	50.473			
36	1.0030	.....	15.638				82	2.1990	.....	56.717			
37	1.0022	.....	15.910				83	2.2049	.....	57.860			
38	1.0029	.....	16.002				84	2.2156	.....	57.986	2.2076	13.685	25.3
39	1.0014	.....	15.951	1.0017	3.7803	13.2	85	2.2062	.....	58.301			
40	1.0012	.....	15.968		(Fence 2)		86	2.2124	.....	57.895			
41	1.0009	.....	15.933				87	2.3622	.....	66.467			
42	1.0041	.....	15.559				88	2.3665	.....	67.729			
43	1.0018	.....	15.525				89	2.3656	.....	66.939	2.3638	15.891	31.3
44	1.0044	.....	15.665	1.0032	3.6964	10.5	90	2.3637	.....	68.418			
45	1.0014	.....	15.618		(Fence 4)		91	2.3610	.....	65.757			
46	1.0044	.....	15.628										

TABLE 15.—SCHODER AND TURNER, SHARP-CRESTED WEIRS, SERIES G.

No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.	No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.
	Float-gauge.	Plumb-bob gauge.						Float-gauge.	Plumb-bob gauge.				
1	0.0176	0.0182	0.0302	0.0179	0.0072	— 8.9	46	1.2112	.....	20.196			
2	0.0306	0.0304	0.0860	0.0305	0.0204	+14.6	47	1.2152	.....	20.306			
3	0.0319	0.0321	0.0972	0.0320	0.0230	20.4	48	1.2109	.....	20.377	1.2154	4.8137	7.9
4	0.0356	0.0352	0.1062	0.0354	0.0252	13.5	49	1.2187	.....	20.415			
5	0.0415	0.0412	0.1318	0.0414	0.0312	11.4	50	1.2150	.....	20.276			
6	0.0477	0.0464	0.1588	0.0470	0.0376	10.9	51	1.4056	.....	25.671			
7	0.0537	0.0532	0.1890	0.0534	0.0448	9.0	52	1.4037	.....	25.712			
8	0.0595	0.0588	0.2190	0.0592	0.0519	8.4	53	1.4031	.....	25.638	1.4043	6.0822	9.8
9	0.0676	0.0663	0.2607	0.0670	0.0618	6.9	54	1.4042	.....	25.616			
10	0.0778	0.0763	0.3180	0.0770	0.0754	5.9	55	1.4051	.....	25.697			
11	0.0883	0.0869	0.3825	0.0876	0.0906	5.0	56	1.6045	.....	31.941			
12	0.0997	0.0992	0.4587	0.0994	0.1087	4.2	57	1.6021	.....	31.768			
13	0.1215	0.1206	0.6102	0.1210	0.1446	3.1	58	1.6007	.....	31.789	1.6016	7.5399	11.7
14	0.1420	0.1408	0.7663	0.1414	0.1816	2.5	59	1.6035	.....	31.953			
15	0.1646	0.1643	0.9568	0.1644	0.2267	2.1	60	1.5954	.....	31.640			
16	0.1820	0.1814	1.1088	0.1817	0.2627	1.9	61	1.8076	.....	38.884			
17	0.1991	0.1984	1.2655	0.1988	0.2999	1.6	62	1.8084	.....	38.766			
18	0.2495	0.2483	1.7733	0.2489	0.4202	1.6	63	1.8075	.....	38.879	1.8081	9.2130	13.8
19	0.3002	0.2992	2.3373	0.2997	0.5539	1.4	64	1.8070	.....	38.862			
20	0.3499	0.3497	2.9572	0.3498	0.7008	1.7	65	1.8098	.....	39.003			
21	0.4014	0.4014	3.6262	0.4014	0.8593	1.5	66	1.9911	.....	45.920			
22	0.4511	0.4500	4.3160	0.4506	1.0227	1.5	67	1.9891	.....	46.011			
23	0.5004	0.4996	5.0917	0.5000	1.2065	2.5	68	1.9974	.....	45.981	1.9936	10.883	16.1
24	0.5003	0.4996	5.0910				69	1.9971	.....	45.871			
25	0.8025	0.8032	10.529	0.8028	2.4957	4.2	70	1.9932	.....	45.857			
26	0.8029	0.8024	10.554				71	2.0011	.....	50.194			
27	0.8023	0.8032	10.513				72	2.0044	.....	49.858			
28	0.9032	0.9049	12.667	0.9041	3.0087	5.1	73	2.0016	.....	49.935	1.9944	11.797 (Fence 2)	25.8
29	0.9028	0.9050	12.700				74	1.9787	.....	49.744			
30	0.9036	0.9051	12.723				75	1.9864	.....	49.188			
31	1.0022	.....	14.955	1.0013	3.5432	6.2	76	1.9899	.....	46.518			
32	1.0019	.....	14.955				77	1.9918	.....	46.625			
33	1.0006	.....	14.949				78	1.9850	.....	46.270	1.9882	11.008 (Fence 4)	17.9
34	1.0015	.....	14.947				79	1.9848	.....	46.413			
35	1.0005	.....	14.953				80	1.9897	.....	46.442			
36	0.9976	.....	15.699	0.9975	3.7209 (Fence 1)	12.2	81	2.1881	.....	53.947			
37	0.9977	.....	15.684				82	2.1883	.....	53.747			
38	0.9969	.....	15.705				83	2.1856	.....	53.643	2.1794	12.682	18.4
39	0.9992	.....	15.723				84	2.1792	.....	53.591			
40	0.9960	.....	15.700				85	2.1537	.....	52.671			
41	1.0036	.....	14.959	1.0031	3.5512 (Fence 3)	6.1	86	2.4101	.....	63.476			
42	1.0036	.....	15.014				87	2.4069	.....	63.658			
43	1.0036	.....	14.967				88	2.4110	.....	63.360	2.4112	15.073	20.9
44	1.0026	.....	14.983				89	2.4108	.....	63.614			
45	1.0021	.....	15.008				90	2.4174	.....	63.937			

TABLE 16.—SCHODER AND TURNER, SHARP-CRESTED WEIRS, SERIES H.

No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.
	Float-gauge.	Plumb-bob gauge.				
1	0.0222	0.0218	0.0478	0.0220	0.0113	3.7
2	0.0342	0.0337	0.1127	0.0340	0.0267	27.8
3	0.0459	0.0451	0.1467	0.0455	0.0348	7.4
4	0.0547	0.0543	0.1915	0.0545	0.0454	7.1
5	0.0611	0.0608	0.2233	0.0607	0.0529	6.2
6	0.0704	0.0694	0.2765	0.0699	0.0655	6.3
7	0.0797	0.0789	0.3283	0.0793	0.0778	4.7
8	0.0892	0.0882	0.3852	0.0887	0.0913	3.8
9	0.1005	0.1002	0.4607	0.1004	0.1092	3.1
10	0.1212	0.1205	0.6033	0.1208	0.1430	2.3
11	0.1402	0.1388	0.7445	0.1395	0.1764	1.7
12	0.1601	0.1591	0.9098	0.1596	0.2156	1.6
13	0.1806	0.1802	1.0867	0.1804	0.2575	0.9
14	0.2002	0.1996	1.2627	0.1999	0.2992	0.5
15	0.2488	0.2472	1.7472	0.2480	0.4140	0.7
16	0.3013	0.3001	2.3353	0.3007	0.5534	0.8
17	0.3506	0.3500	2.9402	0.3503	0.6967	0.9
18	0.4016	0.4007	3.6071	0.4012	0.8548	1.0
19	0.4512	0.4506	4.2986	0.4509	1.0186	1.0
20	0.5025	0.5020	5.0533	0.5024	1.1977	1.0
21	0.5030	0.5021	5.0537			
22	0.8014	0.8026	10.316	0.8019	2.4473	2.3
23	0.8019	0.8019	10.332			
24	0.8018	0.8020	10.335			
25	0.9007	0.9012	12.381	0.9010	2.9368	3.1
26	0.9006	0.9019	12.398			
27	0.9010	0.9008	12.403			
28	1.0024	.....	14.611	1.0025	3.4671	3.7
29	1.0022	.....	14.626			
30	1.0027	.....	14.638			
31	1.0025	.....	14.649			
32	1.0027	.....	14.631			
33	1.0037	.....	14.522			
34	1.0030	.....	14.493			
35	1.0036	.....	14.511	1.0034	3.4429 (Fence 1)	2.9
36	1.0041	.....	14.569			
37	1.0028	.....	14.550			
38	1.0034	.....	15.005			
39	1.0030	.....	15.008			
40	1.0033	.....	14.944	1.0030	3.5500 (Fence 4)	6.1
41	1.0023	.....	15.005			
42	1.0032	.....	14.944			
43	1.2015	.....	19.356	1.2012	4.5886	4.7
44	1.2014	.....	19.352			
45	1.2026	.....	19.351			
46	1.2005	.....	19.395			
47	1.2000	.....	19.365			
48	1.4005	.....	24.703	1.3982	5.8423	6.1
49	1.3960	.....	24.553			
50	1.3988	.....	24.637			
51	1.3992	.....	24.706			
52	1.3965	.....	24.674			
53	1.6148	.....	31.125	1.6127	7.3632	8.0
54	1.6046	.....	30.837			
55	1.6159	.....	31.198			
56	1.6090	.....	30.925			
57	1.6190	.....	31.279			
58	1.8113	.....	37.725	1.8078	8.8850	9.8
59	1.8078	.....	37.453			
60	1.8083	.....	37.369			
61	1.8094	.....	37.528			
62	1.8022	.....	37.401			
63	2.0035	.....	44.224	2.0085	10.550	11.3
64	2.0098	.....	44.445			
65	2.0126	.....	44.758			
66	2.0159	.....	44.815			
67	2.0004	.....	44.357			
68	1.9994	.....	43.148	1.9974	10.215 (Fence 2)	8.7
69	1.9984	.....	43.176			
70	1.9961	.....	43.175			
71	1.9977	.....	42.959			
72	1.9953	.....	43.086			
73	1.9955	.....	49.884	1.9987	11.825 (Fence 3)	25.7
74	1.9974	.....	49.819			
75	2.0040	.....	49.874			
76	2.0000	.....	50.142			
77	1.9965	.....	49.786			
78	2.2052	.....	52.961	2.2043	12.506	14.8
79	2.1987	.....	52.540			
80	2.2053	.....	52.748			
81	2.2033	.....	52.771			
82	2.2088	.....	52.867			
83	2.4680	.....	63.172	2.4746	15.009	15.8
84	2.4716	.....	63.259			
85	2.4758	.....	63.234			
86	2.4791	.....	63.442			
87	2.4787	.....	63.576			

TABLE 17.—SCHODER AND TURNER, SHARP-CRESTED WEIRS, SERIES I.

No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.	No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.
	Float-gauge.	Plumb-bob gauge.						Float-gauge.	Plumb-bob gauge.				
1	0.0224	0.0229	0.0503	0.0226	0.0119	5.3	49	1.8083	.....	35.909			
2	0.0266	0.0265	0.0657	0.0266	0.0156	7.6	50	1.8075	.....	36.105			
3	0.0349	0.0355	0.1058	0.0352	0.0251	14.1	51	1.8116	.....	36.201	1.8097	8.5536	5.5
4	0.0422	0.0425	0.1340	0.0424	0.0318	9.3	52	1.8106	.....	36.096			
5	0.0518	0.0525	0.1808	0.0522	0.0428	7.8	53	1.8107	.....	36.170			
6	0.0602	0.0605	0.2225	0.0604	0.0527	6.7	54	1.9991	.....	42.639			
7	0.0712	0.0714	0.2838	0.0713	0.0673	6.2	55	1.9967	.....	42.503			
8	0.0815	0.0817	0.3412	0.0816	0.0816	5.2	56	2.0004	.....	42.606	2.0009	10.088	7.0
9	0.0902	0.0905	0.3987	0.0904	0.0945	4.4	57	2.0042	.....	42.579			
10	0.1004	0.1004	0.4645	0.1004	0.1101	4.0	58	2.0042	.....	42.538			
11	0.1167	0.1166	0.5752	0.1166	0.1363	2.8	59	2.0147	.....	41.495			
12	0.1404	0.1406	0.7545	0.1405	0.1786	1.8	60	2.0025	.....	41.145			
13	0.1624	0.1626	0.9366	0.1625	0.2219	1.7	61	2.0005	.....	41.331	2.0029	9.7786 (Fence 1)	3.6
14	0.1836	0.1844	1.1192	0.1840	0.2652	0.9	62	1.9985	.....	41.212			
15	0.2010	0.2006	1.2778	0.2008	0.3028	1.1	63	1.9983	.....	41.146			
16	0.2487	0.2485	1.7458	0.2486	0.4137	0.2	64	2.0098	.....	41.890			
17	0.3023	0.3018	2.3483	0.3020	0.5565	0.7	65	2.0055	.....	41.756			
18	0.3517	0.3512	2.9412	0.3514	0.6970	0.5	66	2.0059	.....	41.796	2.0057	9.8945 (Fence 2)	4.6
19	0.4070	0.4070	3.6612	0.4070	0.8676	0.3	67	2.0028	.....	41.621			
20	0.4542	0.4543	4.3078	0.4542	1.0208	0.1	68	2.0043	.....	41.710			
21	0.5048	0.5054	5.0390	0.5050	1.1927	— 0.2	69	2.0086	.....	46.791			
22	0.5045	0.5052	5.0270				70	2.0019	.....	46.687			
23	0.8012	0.8016	10.121	0.8010	2.3970	0.4	71	2.0048	.....	46.907	2.0039	11.094 (Fence 3)	17.5
24	0.8009	0.8014	10.132				72	2.0013	.....	46.825			
25	0.8004	0.8006	10.093				73	2.0029	.....	46.872			
26	0.9026	0.9031	12.203	0.9040	2.8957	1.2	74	2.0036	.....	44.375			
27	0.9031	0.9046	12.196				75	2.0036	.....	44.400			
28	0.9048	0.9058	12.261				76	2.0052	.....	44.284	2.0039	10.516 (Fence 4)	11.3
29	1.0015	.....	14.311	0.9995	3.38.3	1.6	77	2.0045	.....	44.420			
30	0.9979	.....	14.228				78	2.0025	.....	44.410			
31	0.9984	.....	14.254				79	2.2094	.....	49.752			
32	0.9990	.....	14.256				80	2.1972	.....	49.642			
33	1.0058	.....	14.286				81	2.2020	.....	49.653	2.2018	11.757	8.1
34	1.252	.....	18.924				82	2.1982	.....	49.478			
35	1.2424	.....	18.970				83	2.2023	.....	49.544			
36	1.2924	.....	18.973	1.2023	4.4984	2.4	84	2.4129	.....	57.647			
37	1.2931	.....	13.966				85	2.4118	.....	57.461			
38	1.2917	.....	18.957				86	2.4152	.....	57.596	2.4140	13.621	9.1
39	1.2981	.....	23.929				87	2.4165	.....	57.379			
40	1.3974	.....	23.973				88	2.4136	.....	57.318			
41	1.3968	.....	23.965	1.3967	5.6715	3.2	89	2.6661	.....	67.792			
42	1.3952	.....	23.914				90	2.6656	.....	67.631			
43	1.3967	.....	23.888				91	2.6682	.....	67.974	2.6691	16.092	10.8
44	1.3931	.....	29.603				92	2.6678	.....	67.680			
45	1.6078	.....	29.683				93	2.6776	.....	68.462			
46	1.6072	.....	29.861	1.6028	7.0475	4.3							
47	1.6045	.....	29.824										
48	1.6042	.....	29.732										

TABLE 18.—SCHODER AND TURNER, SHARP-CRESTED WEIRS, SERIES J.

No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.	No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.
	Float-gauge.	Plumb-bob gauge.						Float-gauge.	Plumb-bob gauge.				
1	0.0171	0.0169	0.0247	0.0170	0.0059	−20.2	49	1.8006	.....	35.245			
2	0.0260	0.0261	0.0625	0.0260	0.0148	+ 5.7	50	1.7984	.....	35.435			
3	0.0355	0.0360	0.1083	0.0358	0.0257	14.2	51	1.7976	.....	35.259	1.7981	8.3683	4.2
4	0.0445	0.0443	0.1418	0.0444	0.0336	8.0	52	1.7999	.....	35.379			
5	0.0521	0.0520	0.1785	0.0520	0.0423	7.1	53	1.7945	.....	35.253			
6	0.0622	0.0620	0.2302	0.0621	0.0545	5.8	54	2.0015	.....	41.772			
7	0.0696	0.0698	0.2717	0.0697	0.0644	5.1	55	1.9972	.....	41.799			
8	0.0805	0.0809	0.3377	0.0807	0.0800	4.7	56	2.0016	.....	41.948	2.0012	9.9319	5.4
9	0.0888	0.0885	0.3848	0.0886	0.0912	3.9	57	2.0026	.....	41.969			
10	0.1032	0.1034	0.4830	0.1033	0.1145	3.5	58	2.0030	.....	42.075			
11	0.1211	0.1211	0.6068	0.1211	0.1438	2.4	59	1.9983	.....	40.439			
12	0.1393	0.1398	0.7512	0.1396	0.1785	2.8	60	2.0008	.....	40.395			
13	0.1622	0.1618	0.9280	0.1620	0.2199	1.3	61	1.9966	.....	40.410	1.9997	9.5863	1.8
14	0.1788	0.1788	1.0703	0.1788	0.2536	0.7	62	2.0008	.....	40.479		(Fence 1)	
15	0.2026	0.2030	1.2987	0.2028	0.3077	1.2	63	2.0020	.....	40.549			
16	0.2526	0.2526	1.7938	0.2526	0.4251	0.5	64	1.9918	.....	40.101			
17	0.3022	0.3017	2.3372	0.3020	0.5538	0.2	65	1.9941	.....	40.056			
18	0.3507	0.3507	2.9225	0.3507	0.6925	+ 0.1	66	1.9945	.....	40.037	1.9950	9.4939	1.2
19	0.4043	0.4052	3.6110	0.4048	0.8557	− 0.2	67	1.9981	.....	40.081		(Fence 2)	
20	0.4529	0.4534	4.2775	0.4532	1.0136	− 0.4	68	1.9963	.....	40.046			
21	0.5045	0.5052	5.0338				69	1.9941	.....	44.042			
22	0.5049	0.5051	5.0093	0.5049	1.1899	− 0.4	70	1.9986	.....	44.370			
23	0.8001	0.8004	10.014				71	1.9983	.....	44.416	1.9946	10.521	1.5
24	0.8002	0.7999	10.038	0.8000	2.3778	− 0.2	72	1.9990	.....	44.691		(Fence 3)	
25	0.7998	0.7998	10.051				73	1.9830	.....	44.480			
26	0.9019	0.9020	12.037				74	1.9939	.....	42.822			
27	0.9016	0.9030	12.104	0.9020	2.8026	+ 0.3	75	2.0025	.....	42.925			
28	0.9013	0.9022	12.099				76	2.0070	.....	43.212	2.0036	10.198	8.0
29	0.9990	.....	14.151				77	2.0080	.....	43.068		(Fence 4)	
30	0.9991	.....	14.161				78	2.0064	.....	43.158			
31	0.9994	.....	14.170	0.9993	3.3559	0.9	79	2.2033	.....	48.923			
32	0.9991	.....	14.162				80	2.2017	.....	48.777			
33	1.0001	.....	14.165				81	2.2042	.....	48.738	2.2028	11.564	6.2
34	1.2021	.....	18.744				82	2.2036	.....	48.715			
35	1.2021	.....	18.795				83	2.2014	.....	48.854			
36	1.2010	.....	18.797	1.2019	4.4539	1.5	84	2.4061	.....	56.462			
37	1.2018	.....	18.811				85	2.4025	.....	56.240			
38	1.2025	.....	18.831				86	2.4025	.....	56.238	2.4043	13.341	7.5
39	1.4004	.....	23.863				87	2.3990	.....	56.347			
40	1.3993	.....	23.821				88	2.4112	.....	56.215			
41	1.3986	.....	23.798	1.3991	5.6458	2.4	89	2.7001	.....	67.789			
42	1.4003	.....	23.848				90	2.6924	.....	67.587			
43	1.3971	.....	23.796				91	2.7070	.....	68.124	2.6988	16.040	8.6
44	1.6020	.....	29.458				92	2.7033	.....	67 592			
45	1.6056	.....	29.501				93	2.6911	.....	67.361			
46	1.6051	.....	29.516	1.6052	6.9918	3.2							
47	1.6071	.....	29.539										
48	1.6062	.....	29.512										

TABLE 19.—SCHODER AND TURNER, SHARP-CRESTED WEIRS, SERIES K.

No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.	No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.
	Float-gauge.	Plumb-bob gauge.						Float-gauge.	Plumb-bob gauge.				
1	0.0136	0.0137	0.0125	0.0136	0.0030	−43.4	51	1.8067	.....	34.995			
2	0.0190	0.0196	0.0369	0.0193	0.0087	− 2.2	52	1.8055	.....	35.186			
3	0.0279	0.0285	0.0780	0.0282	0.0173	+ 9.5	53	1.8053	.....	35.195	1.8057	8.3300	3.1
4	0.0368	0.0372	0.1143	0.0370	0.0271	14.4	54	1.8044	.....	35.175			
5	0.0461	0.0465	0.1560	0.0463	0.0370	11.4	55	1.8066	.....	35.212			
6	0.0539	0.0545	0.1918	0.0542	0.0455	8.3	56	2.0115	.....	41.597			
7	0.0630	0.0636	0.2410	0.0633	0.0571	7.5	57	2.0116	.....	41.611			
8	0.0709	0.0710	0.2815	0.0710	0.0667	5.9	58	2.0150	.....	41.663	2.0062	9.8371	4.0
9	0.0815	0.0821	0.3482	0.0818	0.0825	5.9	59	2.0082	.....	41.259			
10	0.0930	0.0933	0.4197	0.0932	0.0895	5.1	60	2.0099	.....	41.432			
11	0.0981	0.0985	0.4515	0.0983	0.1070	4.3	61	2.0072	.....	40.110			
12	0.1172	0.1174	0.5822	0.1173	0.1380	3.1	62	2.0109	.....	40.209			
13	0.1390	0.1395	0.7467	0.1392	0.1769	2.3	63	2.0095	.....	40.228	2.0067	9.5078	+ 0.4
14	0.1639	0.1644	0.9508	0.1642	0.2253	1.7	64	2.0029	.....	40.100		(Fence 1)	
15	0.1799	0.1804	1.0873				65	2.0082	.....	39.972			
16	0.1812	0.1809	1.0962	0.1806	0.2587	1.2	66	2.0084	.....	39.709			
17	0.2001	0.2002	1.2717	0.2002	0.3014	1.0	67	2.0000	.....	39.584			
18	0.2515	0.2509	1.7850	0.2512	0.4230	0.9	68	2.0016	.....	39.504	1.9987	9.3869	− 0.2
19	0.3014	0.3012	2.3345	0.3013	0.5532	0.5	69	1.9984	.....	39.646		(Fence 2)	
20	0.3508	0.3507	2.9275	0.3508	0.6937	0.3	70	1.9951	.....	39.620			
21	0.3999	0.4001	3.5583	0.4000	0.8432	0.1	71	1.9977	.....	43.826			
22	0.4509	0.4504	4.2494	0.4506	1.0070	0.0	72	2.0021	.....	44.192			
23	0.4996	0.5000	4.9653				73	1.9981	.....	44.008	1.9992	10.436	+10.9
24	0.4999	0.4997	4.9617	0.4998	1.1762	0.0	74	2.0005	.....	44.232		(Fence 3)	
25	0.8023	.....	10.105				75	1.9974	.....	43.938			
26	0.8021	.....	10.080	0.8022	2.3914	− 0.1	76	1.9951	.....	43.275			
27	0.8021	.....	10.090				77	1.9965	.....	43.253			
28	0.8987	.....	11.987				78	1.9953	.....	43.245	1.9956	10.251	9.2
29	0.8988	.....	11.990	0.8987	2.8412	+ 0.1	79	1.9948	.....	43.278		(Fence 4)	
30	0.8985	.....	11.994				80	1.9961	.....	43.244			
31	0.9989	.....	14.023				81	2.0001	.....	42.058			
32	0.9994	.....	14.078				82	1.9975	.....	42.379			
33	0.9990	.....	14.077	0.9989	3.3354	0.3	83	1.9772	.....	41.892	1.9927	10.005	6.8
34	0.9991	.....	14.110				84	1.9965	.....	42.466		(Fence 5)	
35	0.9983	.....	14.090				85	1.9921	.....	42.330			
36	1.2072	.....	18.571				86	2.2047	.....	48.494			
37	1.2068	.....	18.807				87	2.1970	.....	48.218			
38	1.2072	.....	18.813	1.2071	4.4440	0.6	88	2.2035	.....	48.335	2.2028	11.466	5.3
39	1.2076	.....	18.809				89	2.2044	.....	48.460			
40	1.2065	.....	18.768				90	2.2044	.....	48.435			
41	1.4007	.....	23.619				91	2.4162	.....	56.360			
42	1.3998	.....	23.681				92	2.4090	.....	56.121			
43	1.3968	.....	23.595	1.3989	5.5988	1.6	93	2.4068	.....	55.932	2.4108	13.295	6.7
44	1.3994	.....	23.629				94	2.4095	.....	56.024			
45	1.3976	.....	23.611				95	2.4127	.....	56.085			
46	1.6011	.....	29.302				96	2.6524	.....	65.743			
47	1.6031	.....	29.301				97	2.6489	.....	64.825			
48	1.5996	.....	29.294	1.6017	6.9408	2.8	98	2.6579	.....	65.281	2.6535	15.444	7.3
49	1.6027	.....	29.305				99	2.6555	.....	64.934			
50	1.6021	.....	29.248				100	2.6526	.....	65.085			

TABLE 20.—SCHODER AND TURNER, SHARP-CRESTED WEIRS, SERIES L.

No. of run.	Observed Head. in Feet.		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.	No. of run.	Observed Head. in Feet.		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.
	Float-gauge.	Plumb-bob gauge.						Float-gauge.	Plum bob gauge.				
1	0.0185	0.0187	0.0360	0.0186	0.0085	1.2	20	0.4534	0.4530	4.2958	0.4532	1.0180	0.2
2	0.0277	0.0274	0.0747	0.0276	0.0177	15.7	21	0.5011	0 5000	4.9730	0.5005	1.1788	0.0
3	0.0340	0.0336	0.1060	0.0338	0.0251	21.3	22	0.5008	0.4999	4.9760			
4	0.0453	0.0455	0.1520	0.0454	0.0360	11.8	23	0.9018	.....	12.061			
5	0.0545	0.0543	0.1942	0.0544	0.0460	8.7	24	0.9015	.....	12.066			
6	0.0622	0.0618	0.2378	0.0620	0.0564	9.7	25	0.9024	.....	12.088	0.9019	2.8606	0.3
7	0.0735	0.0731	0.2922	0.0733	0.0709	7.3	26	0.9021	.....	12.083			
8	0.0829	0.0826	0.3552	0.0828	0.0842	6.2	27	0.9016	.....	12.061			
9	0.0909	0.0906	0.4062	0.0908	0.0963	5.7	28	1.5970	.....	29.044			
10	0.1005	0.1000	0.4658	0.1002	0.1104	4.5	29	1.5990	.....	29.008			
11	0.1195	0.1194	0 6027	0.1194	0.1428	3.9	30	1.5965	.....	29.081	1.5968	6.8852	2.5
12	0.1410	0.1406	0.7637	0.1408	0.1810	2.9	31	1.5986	.....	29.081			
13	0.1622	0.1622	0.9433	0.1622	0 2235	2.8	32	1.5931	.....	29.065			
14	0.1832	0.1837	1.1252	0.1834	0.2666	2.0	33	2.2508	.....	50.476			
15	0.2009	0.2004	1.2850	0.2006	0.3045	1.8	34	2.2488	.....	50.427			
16	0.2528	0.2521	1.8030	0.2524	0.4273	1.2	35	2.2387	.....	49.997	2.2451	11.914	6.4
17	0.2980	0.2974	2.3008	0.2977	0.5452	0.8	36	2.2375	.....	50.075			
18	0.3514	0.3507	2.9452	0.3510	0.6979	0 8	37	2.2497	.....	50.409			
19	0.4010	0.4007	3.5793	0.4008	0.8482	0.4							

TABLE 21.—SCHODER AND TURNER, SHARP-CRESTED WEIRS, SERIES M.

No. of run.	Observed Head, in Feet		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.
	Float-gauge.	Plumb-bob gauge.				
1	0.0147	0.0146	0.0188	0.0146	0.0045	−23.7
2	0.0297	0.0296	0.0825	0.0296	0.0195	+15.4
3	0.0347	0.0347	0.1028	0.0347	0.0244	13.5
4	0.0458	0.0457	0.1470	0.0458	0.0348	6.4
5	0.0588	0.0589	0.2125	0.0588	0.0504	6.1
6	0.0695	0.0690	0.2672	0.0692	0.0633	4.3
7	0.0816	0.0814	0.3408	0.0815	0.0818	4.3
8	0.0900	0.0893	0.3927	0.0896	0.0931	4.3
9	0.0981	0.0978	0.4487	0.0980	0.1063	4.0
10	0.1190	0.1196	0.5998	0.1193	0.1421	3.6
11	0.1374	0.1381	0.7390	0.1378	0.1751	2.8
12	0.1569	0.1578	0.8982	0.1574	0.2128	2.3
13	0.1787	0.1795	1.0808	0.1791	0.2561	1.5
14	0.1979	0.1985	1.2552	0.1982	0.2974	1.2
15	0.2466	0.2476	1.7420	0.2471	0.4128	0.9
16	0.2982	0.2989	2.3140	0.2986	0.5483	0.9
17	0.3462	0.3473	2.8942	0.3468	0.6858	0.8
18	0.3981	0.3990	3.5579	0.3986	0.8431	0.6
19	0.4468	0.4471	4.2236	0.4470	1.0013	0.6
20	0.4993	0.5003	4.9817	0.5000	1.1815	0.4
21	0.4999	0.5006	4.9903			
22	0.8029	0.8042	10.118			
23	0.8025	0.8041	10.145	0.8034	2.4024	0.2
24	0.8028	0.8038	10.151			
25	0.8990	0.8991	12.045			
26	0.8987	0.8992	12.041	0.8992	2.8536	0.5
27	0.8998	0.8994	12.040			
28	0.9989	.....	14.093			
29	0.9975	.....	14.138			
30	0.9980	.....	14.128	0.9981	3.3160	0.8
31	0.9978	.....	14.121			
32	0.9985	.....	14.121			
33	1.1995	.....	18.727			
34	1.1992	.....	18.722			
35	1.1974	.....	18.678	1.1983	4.4334	1.5
36	1.1971	.....	18.703			
37	1.1984	.....	18.715			

No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.
	Float-gauge.	Plumb-bob gauge.				
38	1.4057	.....	23.880			
39	1.4068	.....	23.959			
40	1.4079	.....	23.961	1.4065	5.6731	2.1
41	1.4067	.....	23.973			
42	1.4056	.....	23.929			
43	1.5938	.....	29.222			
44	1.5956	.....	29.142			
45	1.5952	.....	29.102	1.5948	6.8997	2.9
46	1.5975	.....	29.115			
47	1.5919	.....	29.003			
48	1.8046	.....	35.142			
49	1.7996	.....	34.967			
50	1.8015	.....	35.111	1.8019	8.3098	3.2
51	1.8019	.....	35.057			
52	1.8021	.....	35.060			
53	1.9994	.....	41.283			
54	1.9940	.....	41.159			
55	1.9937	.....	41.143	1.9954	9.7606	4.0
56	1.9921	.....	41.148			
57	1.9978	.....	41.215			
58	2.2131	.....	48.293			
59	2.2173	.....	48.299			
60	2.2142	.....	48.345	2.2136	11.456	4.5
61	2.2148	.....	48.333			
62	2.2086	.....	48.457			
63	2.4060	.....	55.352			
64	2.4141	.....	55.477			
65	2.4039	.....	55.300	2.4084	13.125	5.4
66	2.4061	.....	55.435			
67	2.4117	.....	55.383			
68	2.5201	.....	59.660			
69	2.5311	.....	59.675			
70	2.5284	.....	59.437	2.5272	14.136	5.7
71	2.5308	.....	59.624			
72	2.5256	.....	59.864			

TABLE 22.—SCHODER AND TURNER, SHARP-CRESTED WEIRS, SERIES N.

No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.	No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.
	Float-gauge.	Plumb-bob gauge.						Float-gauge.	Plumb-bob gauge.				
1	0.0120	0.0126	0.0055	0.0123	0.0013	+71.8	51	0.9975	.....	16.414			
2	0.0217	0.0220	0.0458	0.0218	0.0109	+ 1.9	52	0.9993	.....	16.491			
3	0.0293	0.0296	0.0797	0.0294	0.0189	12.5	53	1.0002	.....	16.547	0.9986	3.9071 (Fence 2)	17.6
4	0.0342	0.0343	0.0995	0.0342	0.0236	11.8	54	0.9994	.....	16.509			
5	0.0422	0.0428	0.1298	0.0425	0.0308	5.5	55	0.9968	.....	16.481			
6	0.0416	0.0415	0.1262	0.0416	0.0299	5.7	56	1.2070	.....	21.711			
7	0.0423	0.0424	0.1297	0.0424	0.0307	5.5	57	1.2078	.....	21.808			
8	0.0512	0.0510	0.1707	0.0511	0.0405	5.2	58	1.2075	.....	21.782	1.2076	5.1609	16.8
9	0.0611	0.0611	0.2227	0.0611	0.0528	5.0	59	1.2084	.....	21.770			
10	0.0613	0.0609	0.2253	0.0611	0.0534	6.2	60	1.2073	.....	21.822			
11	0.0684	0.0680	0.2638	0.0682	0.0625	5.4	61	1.4118	.....	28.349			
12	0.0686	0.0683	0.2660	0.0684	0.0630	5.7	62	1.4051	.....	28.201			
13	0.0790	0.0788	0.3257	0.0789	0.0772	4.6	63	1.4038	.....	28.205	1.4078	6.7031	20.5
14	0.0902	0.0902	0.3937	0.0902	0.0933	3.4	64	1.4101	.....	28.358			
15	0.1016	0.1017	0.4705	0.1016	0.1115	3.3	65	1.4084	.....	28.320			
16	0.1217	0.1217	0.6132	0.1217	0.1454	2.8	66	1.6053	.....	35.067			
17	0.1412	0.1411	0.7645	0.1412	0.1812	2.5	67	1.6002	.....	35.108			
18	0.1591	0.1588	0.9153	0.1590	0.2169	2.7	68	1.6037	.....	35.254	1.6040	8.3491	23.4
19	0.1799	0.1800	1.1013	0.1800	0.2610	2.6	69	1.6064	.....	35.331			
20	0.2005	0.2004	1.2928	0.2004	0.3064	2.6	70	1.6043	.....	35.407			
21	0.2022	0.2023	1.3010	0.2022	0.3083	1.8	71	1.8572	.....	45.126			
22	0.2535	0.2536	1.8518	0.2536	0.4388	3.2	72	1.8578	.....	44.726			
23	0.2989	0.2986	2.3800	0.2988	0.5640	3.7	73	1.8492	.....	44.472	1.8493	10.537	25.8
24	0.3490	0.3488	3.0148	0.3489	0.7144	4.1	74	1.8396	.....	44.146			
25	0.4014	0.4015	3.7436	0.4014	0.8871	4.7	75	1.8328	.....	43.862			
26	0.4534	0.4536	4.5073				76	2.0094	.....	51.870			
27	0.4534	0.4541	4.5407	0.4536	1.0727	5.5	77	2.0122	.....	51.987			
28	0.4537	0.4530	4.5800				78	2.0042	.....	51.457	2.0087	12.266	29.4
29	0.4537	0.4536	4.5283				79	2.0126	.....	52.005			
30	0.5006	0.5009	5.2617	0.5006	1.2482	5.8	80	2.0051	.....	51.490			
31	0.5005	0.5006	5.2730				81	2.0040	.....	54.017			
32	0.7030	.....	9.0230				82	1.9908	.....	53.595			
33	0.7034	.....	9.0663	0.7033	2.1453	9.2	83	1.9998	.....	53.401	2.0009	12.692 (Fence 1)	34.7
34	0.7036	.....	9.0705				84	2.0032	.....	53.317			
35	0.8033	.....	11.234				85	2.0067	.....	53.480			
36	0.8042	.....	11.255	0.8038	2.6647	11.0	86	2.0020	.....	50.167			
37	0.8039	.....	11.246				87	2.0078	.....	50.460			
38	0.9031	.....	13.598				88	2.0088	.....	50.505	2.0068	11.950 (Fence 2)	26.2
39	0.9030	.....	13.589	0.9029	3.2204	12.7	89	2.0067	.....	50.343			
40	0.9026	.....	13.584				90	2.0092	.....	50.661			
41	1.0031	.....	16.122				91	2.1968	.....	60.354			
42	1.0024	.....	16.143				92	2.2064	.....	60.740			
43	1.0033	.....	16.118	1.0031	3.8228	+11.3	93	2.2014	.....	60 588	2.2012	14.316	31.6
44	1.0036	.....	16.142				94	2.2011	.....	60.405			
45	1.0030	.....	16.137				95	2.2002	.....	59.979			
46	1.0084	.....	16.458				96	2.3336	.....	67.666			
47	1.0074	.....	16.492				97	2.3341	.....	67.987			
48	1.0029	.....	16.466	1.0069	3.9047	16.1	98	2.3368	.....	67.925	2.3301	16.062	35.6
49	1.0080	.....	16.484		(Fence 1)		99	2.3293	.....	67.604			
50	1.0080	.....	16.489				100	2.3166	.....	67.724			

TABLE 23.—Schoder and Turner, Sharp-Crested Weirs, Series O.

No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.	No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.
	Float-gauge.	Plumb-bob gauge.						Float-gauge.	Plumb-bob gauge.				
1	0.0511	0.0515	0.1700	0.0513	0.0408	+4.1	40	1.6062	.....	29.348			
2	0.0509	0.0509	0.1685	0.0509	0.0399	4.2	41	1.6003	.....	29.301			
3	0.1013	0.1016	0.4650	0.1014	0.1102	2.5	42	1.6001	.....	29.234	1.6024	6.9478	+2.9
4	0.1016	0.1016	0.4585	0.1016	0.1086	0.6	43	1.6026	.....	29.396			
5	0.1219	0.1219	0.6102	0.1219	0.1446	2.0	44	1.6028	.....	29.319			
6	0.1406	0.1411	0.7497	0.1408	0.1777	0.9	45	1.8138	.....	35.734			
7	0.1404	0.1401	0.7417	0.1402	0.1758	0.6	46	1.8128	.....	35.685			
8	0.1612	0.1603	0.9123	0.1608	0.2162	0.7	47	1.8085	.....	35.483	1.8130	8.4562	4.0
9	0.1802	0.1799	1.0817	0.1800	0.2563	0.8	48	1.8136	.....	35.716			
10	0.2013	0.2016	1.2720	0.2014	0.3014	0.1	49	1.8163	.....	35.807			
11	0.2012	0.2009	1.2708	0 2010	0.3010	0.3	50	2.0037	.....	41.841			
12	0.2508	0.2501	1.7705	0.2504	0.4196	0.6	51	2.0055	.....	42 044			
13	0.2988	0.2981	2.3032	0.2984	0.5458	+0.6	52	2.0065	.....	41.969	2.0040	9.9255	5.1
14	0.3510	0.3502	2.9333	0.3506	0.6951	+0.5	53	2.0040	.....	42.015			
15	0.4002	0.4000	3.5560	0.4001	0.8427	0.0	54	2.0003	.....	41.559			
16	0.4530	0.4528	4.2761	0.4529	1.0133	−0.2	55	2.1961	.....	48.597			
17	0.5015	0.5012	4.9960	0.5013	1.1846	+0.2	56	2.1937	.....	48.357			
18	0.5015	0.5011	5.0023				57	2.1964	.....	48.443	2.1948	11.480	6.0
19	0.8007	.....	10.073				58	2.1940	.....	48.379			
20	0.8008	.....	10.102	0.8009	2.3911	+0.2	59	2.1937	.....	48.460			
21	0.8011	.....	10.096				60	2.3906	.....	55.539			
22	0.9014	.....	12.114				61	2.3978	.....	55.891			
23	0.9018	.....	12.110	0.9019	2.8705	+0.6	62	2.4006	.....	56.215	2.4040	13.298	7.1
24	0.9024	.....	12.117				63	2.4138	.....	56.419			
25	1.0018	.....	14.184				64	2.4170	.....	56.584			
26	1.0014	.....	14.193				65	2.5736	.....	62.996			
27	1.0019	.....	14.189	1.0016	3.3618	+0.7	66	2.5690	.....	62.519			
28	1.0020	.....	14.190				67	2.5570	.....	62.627	2.5704	14.880	8.4
29	1.0008	.....	14.178				68	2.5710	.....	62.901			
30	1.2029	.....	18.716				69	2.5816	.....	62.920			
31	1.2023	.....	18.811				70	2.7118	.....	68.636			
32	1.2036	.....	18.818	1.2026	4.4506	+1.3	71	2.7142	.....	68.423			
33	1.2018	.....	18.793				72	2.7126	.....	68.338	2.7090	16.217	9.2
34	1.2024	.....	18.770				73*	2.6876	.....	68.962			
35	1.4009	.....	23.678				74	2.6976	.....	68.344			
36	1.3997	.....	23.780										
37	1.4035	.....	23.843	1.4007	5.6376	+2.1							
38	1.3973	.....	23.774										
39	1.4023	.....	23.879										

*Omit in average.

TABLE 24.—Schoder and Turner, Sharp-Crested Weirs, Series P.

No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.	No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.
	Float-gauge.	Plumb-bob gauge.						Float-gauge.	Plumb-bob gauge.				
1	1.0045	.....	14.213				7	2.0442	.....	42.847			
2	1.0095	.....	14.389	1.0075	3.3976	+0.9	8	2.0404	.....	42.625	2.0413	10.1301	4.3
3	1.0085	.....	14.413				9	2.0392	.....	42.775			
4	1.0086	.....	14.617				10	2.0422	.....	44.588			
5	1.0075	.....	14.567	1.0102	3.4732	2.7	11	2.0465	.....	45.034	2.0416	10.6524	9.7
6	1.0144	.....	14.788				12	2.0362	.....	45.237			

TABLE 25.—Dawson, (Nearly) Sharp-Crested Weir, Series I.

No. of run.	Observed head, in feet, by float-gauge.	Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.	No. of run.	Observed head, in feet, by float-gauge.	Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.
1	0.096	0.4472	0.096	0.1060	7.1	21	1.3955	23.79			
2	0.187	1.185	0.186	0.2819	5.5	22	1.397	23.80	1.397	5.642	2.6
3	0.185	1.194				23	1.398	23.83			
4	0.267	2.006	0.267	0.4778	4.0	24	1.613	29.87			
5	0.267	2.027				25	1.612	29.88	1.612	7.078	3.9
6	0.443	4.249	0.443	1.007	2.5	26	1.811	35.31			
7	0.459	4.477	0.459	1.061	2.4	27	1.806	35.29	1.813	8.416	3.5
8	0.542	5.752	0.542	1.363	2.6	28	1.818	35.78			
9	0.559	6.035	0.559	1.430	2.7	29	1.817	35.69			
10	0.789	10.05				30	2.021	42.27			
11	0.789	10.01	0.789	2.380	2.0	31	2.023	42.30	2.023	10.021	4.6
12	0.789	10.04				32	2.025	42.28			
13	0.910	12.39				33	2.398	54.51			
14	0.910	12.43	0.910	2.940	1.7	34	2.398	55.15	2.394	12.979	5.2
15	1.000	14.31				35	2.403	54.93			
16	1.001	14.40	1.000	3.408	2.2	36	2.376	53.50			
17	1.000	14.37				37	2.453	58.03	2.453	13.751	7.5
18	1.203	19.06				38	2.543	62.00			
19	1.202	18.86	1.202	4.498	2.5	39	2.548	59.10	2.546	14.349	6.1
20	1.2005	19.02									

TABLE 26.—Dawson, Sharp-Crested Weir, Series II.

No. of run.	Observed head, by float-gauge, in feet.	Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.	No. of run.	Observed head, by float-gauge, in feet.	Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.
1	0.111	0.5348				27	1.212	18.829			
2	0.111	0.5366	0.1110	0.1271	3.3	28	1.213	18.799	1.2123	4.4600	0.3
3	0.111	0.5375				29	1.212	18.834			
4	0.177	1.0472				30	1.372	22.748			
5	0.177	1.0500	0.1770	0.2488	0.3	31	1.373	22.880	1.3720	5.3984	0.8
6	0.177	1.0630				32	1.371	22.651			
7	0.248	1.7330				33	1.565	27.924			
8	0.248	1.7383	0.2480	0.4116	+0.1	34	1.568	27.919	1.5690	6.6275	1.3
9	0.248	1.7395				35	1.574	28.061			
10	0.317	2.5062				36	1.760	33.418			
11	0.317	2.5110	0.3170	0.5925	−0.3	37	1.761	33.558	1.7597	7.9870	2.1
12	0.317	2.5143				38	1.758	33.506			
13	0.425	3.8788				39	1.915	38.614			
14	0.425	3.8563	0.4250	0.9163	−0.7	40	1.917	38.702	1.9187	9.1600	3.5
15	0.425	3.8661				41	1.924	38.648			
16	0.546	5.6860				42	2.155	46.233			
17	0.546	5.6737	0.5460	1.3460	+0.2	43	2.151	46.101	2.1537	10.936	3.9
18	0.546	5.6503				44	2.155	46.119			
19	0.546	5.6908				45	2.352	53.015			
20	0.926	12.461				46	2.349	53.407	2.3472	12.578	5.0
21	0.928	12.538	0.9288	2.9645	−0.5	47	2.345	52.736			
22	0.927	12.382				48	2.343	53.158			
23	0.934	12.660				49	2.645	64.928			
24	1.050	15.125				50	2.652	64.983			
25	1.057	15.177	1.0520	3.5870	−0.2	51	2.655	65.465	2.6496	15.402	7.2
26	1.049	15.108				52	2.647	64.799			
						53	2.649	64.809			

TABLE 27.—DAWSON, SHARP-CRESTED WEIR (AFTER SIX WEEKS' SERVICE), SERIES III.

No. of run.	Observed head by float-gauge, in feet.	Measured discharge, in second-feet, per foot of weir.	Mean of observed heads, in feet.	Mean discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.	No. of run.	Observed head by float-gauge, in feet.	Measured discharge, in second-feet per foot of weir.	Mean of observed heads, in feet.	Mean discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.
1	0.115	0.1360				18	1.079	3.754			
2	0.115	0.1348	0.115	0.1355	+4.3	19	1.079	3.757	1.079	3.755	0.6
3	0.115	0.1358				20	1.079	3.754			
4	0.254	0.4306				21	1.368	5.39			
5	0.254	0.4310	0.254	0.4310	1.1	22	1.366	5.405	1.3655	5.395	1.5
6	0.254	0.4314				23	1.363	5.385			
7	0.330	0.638				24	1.365	5.40			
8	0.330	0.636	0.330	0.6360	0.7	25	1.680	7.425			
9	0.330	0.634				26	1.678	7.430			
10	0.437	0.960				27	1.683	7.420	1.6802	7.424	2.3
11	0.437	0.967	0.437	0.9660	0.3	28	1.680	7.420			
12	0.437	0.971				29	2.082	10.26			
13	0.524	1.281				30	2.086	10.26			
14	0.524	1.281	0.524	1.2810	1.5	31	2.087	10.44	2.0836	10.340	3.2
15	0.978	3.225				32	2.082	10.37			
16	0.978	3.223	0.978	3.2250	0.1	33	2.081	10.37			
17	0.978	3.227									

TABLE 28.—MARTIN, SHARP-CRESTED WEIRS.

No. of run.	Observed head, mean of point and plumb-bob gauges, in feet.	Measured discharge, in second-feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.	No. of run.	Observed head, mean of point and plumb-bob gauges, in feet.	Measured discharge, in second-feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.
1	0.095	0.2050	0.1013	3.9	16	0.693	4.130	2.042	6.2
2	0.151	0.4167	0.2060	5.3	17	0.717	4.357	2.154	6.5
3	0.204	0.6352	0.3140	2.3	18	0.741	4.560	2.254	6.0
4	0.204	0.6302	0.3115	1.5	19	0.812	5.312	2.626	7.7
5	0.239	0.8109	0.4008	3.0	20	0.851	5.711	2.823	7.9
6	0.306	1.172	0.5793	2.7	21	0.861	5.777	2.856	7.3
7	0.339	1.368	0.6738	2.4	22	0.874	5.973	2.952	8.5
8	0.400	1.753	0.8665	2.8	23	0.910	6.341	3.134	8.4
9	0.459	2.177	1.076	3.9	24	0.910	6.349	3.138	8.5
10	0.491	2.406	1.189	3.7	25	0.914	6.424	3.176	9.1
11	0.560	2.940	1.453	4.1	26	0.915	6.431	3.179	9.0
12	0.571	3.059	1.512	5.2	27	0.960	6.988	3.454	10.2
13	0.595	3.205	1.584	3.6	28	1.029	7.676	3.795	9.1
14	0.631	3.572	1.766	5.7	29	1.052	8.000	3.954	10.0
15	0.640	3.667	1.813	6.3	30	1.064	8.032	3.970	8.6

TABLE 29.—MEYER AND SEE, SHARP-CRESTED WEIRS, SET I, SERIES A.
(Runs, 1 to 42, 43 to 56, 57 to 106, 107 to 109; Date, March 21, March 26, April 2, and May 7, 1914; Water Temperature, in Degrees Fahrenheit, 33, 34, 38, 62.)

No. of run.	Observed Head, in Feet.		Measured discharge in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet per foot of weir.	Percentage excess over discharge by Francis' formula.	No. of run.	Observed Head, in Feet.		Measured discharge in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.
	Float No. 1.	Float No. 2.						Float No. 1.	Float No. 2.				
1	0.050	0.051	0.0858	0.0505	0.0423	11.8	54	.....	.....	1.9356	0.430	0.9554	1.7
2	0.050	0.051	0.0861	0.0505	0.0425	12.1	55	.....	.....	1.9313	0.430	0.9533	1.5
3	.....	.....	0.1057	0.060	0.0522	6.6	56	.....	.....	1.9324	0.430	0.9538	1.6
4	.....	.....	0.1055	0.060	0.0521	6.5	57	0.440	0.441	1.9988	0.4405	0.9866	1.3
5	0.070	0.071	0.1404	0.0705	0.0693	11.4	58	.....	.....	2.0663	0.450	1.0199	1.5
6	0.070	0.071	0.1379	0.0705	0.0681	9.2	59	.....	.....	2.0782	0.450	1.0258	2.1
7	0.080	0.081	0.1668	0.0805	0.0823	8.4	60	.....	.....	2.2154	0.470	1.0935	1.8
8	0.080	0.081	0.1660	0.0805	0.0819	7.9	61	.....	.....	2.2150	0.470	1.0933	1.8
9	0.091	0.093	0.2045	0.0920	0.1009	8.7	62	.....	.....	2.3239	0.486	1.1470	1.8
10	0.092	0.094	0.2088	0.0930	0.1006	6.3	63	.....	.....	2.3679	0.490	1.1687	2.3
11	.....	.....	0.2194	0.0980	0.1083	6.0	64	.....	.....	2.3647	0.490	1.1672	2.2
12	.....	.....	0.2194	0.0980	0.1083	6.0	65	.....	.....	2.5070	0.510	1.2374	2.1
13	0.111	0.113	0.2706	0.1120	0.1336	6.9	66	.....	.....	2.5145	0.510	1.2411	2.3
14	0.111	0.113	0.2694	0.1120	0.1330	6.5	67	.....	.....	2.8088	0.550	1.3864	2.1
15	.....	.....	0.2901	0.1190	0.1432	4.7	68	.....	.....	2.7985	0.549	1.3813	2.1
16	0.130	0.131	0.3303	0.1305	0.1630	3.9	69	0.580	0.582	3.0414	0.581	1.5012	2.0
17	0.130	0.131	0.3302	0.1305	0.1630	3.9	70	.....	.....	3.1806	0.598	1.5699	2.0
18	.....	.....	0.3554	0.1370	0.1754	3.9	71	.....	.....	3.1893	0.599	1.5742	2.0
19	0.150	0.152	0.4141	0.1510	0.2044	4.5	72	0.649	0.647	3.6040	0.648	1.7789	2.4
20	0.150	0.152	0.4148	0.1510	0.2047	4.7	73	.....	.....	3.6576	0.654	1.8053	2.4
21	.....	.....	0.4609	0.1640	0.2275	2.6	74	0.703	0.702	4.0571	0.7025	2.0025	2.1
22	0.170	0.171	0.4893	0.1705	0.2415	3.1	75	0.699	0.703	4.0424	0.701	1.9952	2.1
23	0.170	0.171	0.4880	0.1705	0.2409	2.6	76	0.730	0.735	4.3147	0.7325	2.1296	2.0
24	0.190	0.191	0.5760	0.1905	0.2843	2.6	77	.....	.....	4.4801	0.751	2.2113	2.1
25	0.190	0.191	0.5771	0.1905	0.2848	2.9	78	0.750	0.749	4.4577	0.7495	2.2002	1.8
26	.....	.....	0.6130	0.1990	0.3026	2.4	79	.....	.....	4.7146	0.775	2.3270	2.4
27	0.210	0.211	0.6738	0.2105	0.3326	3.4	80	0.798	0.801	4.9048	0.7995	2.4209	1.8
28	0.210	0.211	0.6685	0.2105	0.3300	2.6	81	0.798	0.799	4.9109	0.7985	2.4239	2.1
29	0.230	0.231	0.7643	0.2305	0.3772	2.4	82	0.810	0.814	5.0427	0.812	2.4890	2.1
30	0.230	0.231	0.7598	0.2305	0.3750	1.8	83	0.850	0.851	5.3925	0.8505	2.6616	2.0
31	.....	.....	0.8003	0.2380	0.3950	2.3	84	0.849	0.848	5.3990	0.8485	2.6648	2.3
32	0.250	0.251	0.8618	0.2505	0.4254	2.1	85	0.886	0.891	5.7756	0.8885	2.8507	2.3
33	0.250	0.251	0.8648	0.2505	0.4268	2.4	86	0.904	0.907	5.9325	0.9055	2.9282	2.1
34	0.270	0.271	0.9702	0.2705	0.4789	2.4	87	0.904	0.907	5.9406	0.9055	2.9322	2.1
35	0.271	0.272	0.9732	0.2715	0.4804	2.3	88	0.947	0.950	6.3944	0.9485	3.1561	2.6
36	.....	.....	1.0115	0.279	0.4993	2.0	89	0.950	0.951	6.4160	0.9505	3.1668	2.4
37	.....	.....	1.0736	0.290	0.5299	1.8	90	.....	.....	6.4236	0.950	3.1706	2.9
38	.....	.....	1.0689	0.290	0.5276	1.5	91	0.999	1.003	6.9901	1.001	3.4502	3.4
39	.....	.....	1.1841	0.310	0.5844	1.8	92	0.999	1.001	6.9604	1.000	3.4355	3.2
40	.....	.....	1.1830	0.310	0.5839	1.6	93	1.054	1.059	7.5326	1.0565	3.7179	2.9
41	.....	.....	1.2411	0.320	0.6126	1.6	94	1.104	1.106	8.0572	1.105	3.9769	2.9
42	.....	.....	1.2985	0.330	0.6409	1.5	95	1.099	1.105	8.0569	1.102	3.9767	3.2
43	.....	.....	1.2978	0.330	0.6406	1.5	96	1.134	1.138	8.4023	1.136	4.1472	2.9
44	.....	.....	1.4232	0.350	0.7025	1.8	97	1.196	1.202	9.2059	1.199	4.5438	3.9
45	.....	.....	1.4256	0.350	0.7036	2.0	98	1.200	1.198	9.1638	1.199	4.5231	3.4
46	.....	.....	1.4692	0.358	0.7252	1.6	99	1.249	1.257	9.8722	1.253	4.8727	4.4
47	.....	.....	1.5410	0.370	0.7606	1.5	100	1.257	1.261	9.8656	1.259	4.8695	3.4
48	.....	.....	1.5379	0.370	0.7591	1.3	101	1.303	1.300	10.4234	1.3015	5.1448	4.0
49	.....	.....	1.6707	0.390	0.8246	1.7	102	1.300	1.302	10.3757	1.301	5.1212	3.7
50	.....	.....	1.6702	0.390	0.8244	1.7	103	1.301	1.307	10.5625	1.304	5.2134	5.0
51	0.400	0.401	1.7335	0.4005	0.8556	1.3	104	1.334	1.330	11.0884	1.332	5.4730	7.1
52	.....	.....	1.8013	0.410	0.8891	1.6	105	1.334	1.339	11.1675	1.3365	5.5121	7.3
53	.....	.....	1.8090	0.410	0.8929	2.1	106	1.338	1.344	11.1373	1.341	5.4971	6.3

TABLE 30.—MEYER AND SEE, SHARP-CRESTED WEIRS, SET I, SERIES B.
(Date, May 11-12, 1914; Temperature of Water, 56 to 57° Fahr.)

No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge, by Francis' formula.	No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.
	Float No. 1.	Float No. 2.						Float No. 1.	Float No. 2.				
1	.....	.....	0.0717	0.043	0.0855	19.5	20	0.393	0.392	1.6860	0.3925	0.8342	1.9
2	.....	.....	0.1182	0.059	0.0585	22.6	21	.....	.....	1.8861	0.422	0.9333	2.2
3	.....	.....	0.1832	0.084	0.0906	11.7	22	.....	.....	2.1433	0.459	1.0605	2.4
4	.....	.....	0.2370	0.102	0.1173	8.1	23	.....	.....	2.4676	0.503	1.2210	2.8
5	0.119	0.118	0.2911	0.1185	0.1440	6.0	24	.....	.....	2.9236	0.547	1.3971	3.7
6	.....	.....	0.3711	0.140	0.1836	5.3	25	.....	.....	3.2446	0.599	1.6054	4.0
7	.....	.....	0.3856	0.143	0.1908	5.9	26	.....	.....	3.6550	0.649	1.8085	3.9
8	.....	.....	0.4443	0.159	0.2198	4.1	27	.....	.....	3.8397	0.673	1.8969	3.3
9	.....	.....	0.5320	0.179	0.2632	4.4	28	.....	.....	4.0897	0.703	2.0236	3.1
10	.....	.....	0.5844	0.190	0.2892	2.2	29	.....	.....	4.4859	0.747	2.2196	3.2
11	.....	.....	0.7142	0.219	0.3534	3.5	30	.....	.....	4.9850	0.804	2.4666	2.7
12	.....	.....	0.8050	0.239	0.3983	2.4	31	.....	.....	5.4476	0.849	2.6955	3.5
13	.....	.....	0.9268	0.262	0.4586	2.7	32	.....	.....	5.9107	0.897	2.9246	3.4
14	.....	.....	1.0119	0.278	0.5007	2.6	33	.....	.....	6.5428	0.957	3.2374	3.8
15	.....	.....	1.1263	0.299	0.5573	2.4	34	.....	.....	7.0000	1.001	3.4636	3.9
16	.....	.....	1.2474	0.320	0.6172	2.4	35	.....	.....	8.0897	1.101	4.0026	4.0
17	.....	.....	1.3737	0.341	0.6797	2.5	36	.....	.....	9.2257	1.197	4.5649	4.7
18	.....	.....	1.4682	0.359	0.7265	1.4	37	.....	.....	9.9301	1.257	4.9135	4.7
19	.....	.....	1.6013	0.379	0.7923	2.0	38	.....	.....	11.0740	1.325	5.4795	7.9

TABLE 31.—MEYER AND SEE, SHARP-CRESTED WEIRS, SET I, SERIES C.
(Date, May 14, 1914; Temperature of Water, 50° Fahr.)

No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.	No. of run.	Observed Head. in Feet.		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.
	Float No. 1.	Float No. 2.						Float No. 1.	Float No. 2.				
1	.....	.....	0.0992	0.053	0.0491	20.9	20	.....	.....	1.8756	0.421	0.9281	2.0
2	.....	.....	0.1053	0.058	0.0521	12.0	21	.....	.....	2.1085	0.451	1.0433	3.4
3	.....	.....	0.1507	0.071	0.0746	18.4	22	.....	.....	2.4642	0.499	1.2193	3.9
4	.....	.....	0.2320	0.094	0.1148	19.6	23	.....	.....	2.8178	0.546	1.3943	3.8
5	.....	.....	0.2618	0.110	0.1295	6.6	24	.....	.....	3.2401	0.599	1.6032	3.5
6	.....	.....	0.3351	0.131	0.1658	5.0	25	.....	.....	3.2922	0.608	1.6290	3.2
7	.....	.....	0.3476	0.134	0.1720	5.3	26	.....	.....	3.6761	0.650	1.8190	4.3
8	.....	.....	0.4165	0.152	0.2061	4.5	27	.....	.....	4.1030	0.701	2.0302	3.9
9	.....	.....	0.4847	0.169	0.2398	3.6	28	.....	.....	4.5781	0.754	2.2653	3.9
10	.....	.....	0.5842	0.191	0.2891	4.0	29	.....	.....	5.0265	0.802	2.4871	4.0
11	.....	.....	0.6862	0.214	0.3395	3.0	30	.....	.....	5.5027	0.853	2.7228	3.8
12	.....	.....	0.7722	0.232	0.3821	2.7	31	.....	.....	6.0119	0.904	2.9747	3.9
13	.....	.....	0.8703	0.251	0.4306	2.8	32	.....	.....	6.5260	0.953	3.2291	4.2
14	0.270	0.269	0.9642	0.2695	0.4791	2.8	33	.....	.....	7.0652	1.003	3.4959	4.5
15	.....	.....	1.0825	0.291	0.5356	2.5	34	.....	.....	8.2791	1.114	4.0965	4.6
16	.....	.....	1.2526	0.321	0.6198	2.3	35	.....	.....	9.2991	1.198	4.6012	5.4
17	.....	.....	1.4363	0.352	0.7107	2.2	36	.....	.....	10.0257	1.260	4.9608	5.3
18	.....	.....	1.5539	0.371	0.7689	2.2	37	.....	.....	11.0673	1.328	5.4762	7.5
19	.....	.....	1.6817	0.384	0.8074	1.9							

TABLE 32.—MEYER AND SEE, SHARP-CRESTED WEIRS, SET I, SERIES D.
(Date, May 16, 1914; Temperature of Water, 52 to 53° Fahr.)

No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.	No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.
	Float No. 1.	Float No. 2.						Float No. 1.	Float No. 2.				
1	.....	.....	0.0879	0.048	0.0400	14.3	20	.....	.....	1.7887	0.410	0.8855	1.3
2	0.069	0.070	0.1412	0.0695	0.0699	14.6	21	.....	.....	2.0650	0.447	1.0223	2.7
3	.....	.....	0.1535	0.077	0.0760	6.7	22	.....	.....	2.4167	0.497	1.1964	2.5
4	.....	.....	0.2520	0.101	0.1248	16.8	23	.....	.....	2.4884	0.505	1.2319	3.1
5	.....	.....	0.3338	0.122	0.1652	16.4	24	.....	.....	2.8182	0.548	1.3951	3.3
6	.....	.....	0.3532	0.137	0.1749	3.6	25	.....	.....	3.2962	0.606	1.6318	3.9
7	.....	.....	0.4072	0.139	0.2016	16.8	26	.....	.....	3.6944	0.654	1.8289	3.8
8	.....	.....	0.4729	0.167	0.2341	3.0	27	.....	.....	4.1567	0.709	2.0578	3.5
9	.....	.....	0.5441	0.184	0.2694	2.5	28	.....	.....	4.6179	0.762	2.2861	3.2
10	.....	.....	0.6397	0.205	0.3167	2.5	29	.....	.....	5.1550	0.818	2.5520	3.6
11	.....	.....	0.7045	0.219	0.3488	2.2	30	.....	.....	5.6563	0.869	2.8002	3.8
12	.....	.....	0.7985	0.239	0.3953	1.6	31	0.918	0.920	6.1700	0.919	3.0545	4.1
13	.....	.....	0.9088	0.260	0.4499	1.9	32	.....	.....	6.8356	0.973	3.3840	5.9
14	.....	.....	1.0307	0.283	0.5102	1.8	33	.....	.....	7.5190	1.044	3.7223	4.8
15	.....	.....	1.0272	0.283	0.5085	1.4	34	.....	.....	8.2847	1.113	4.1013	4.9
16	.....	.....	1.1214	0.300	0.5551	1.4	35	1.217	1.219	9.5103	1.218	4.7081	5.2
17	.....	.....	1.2539	0.323	0.6207	1.5	36	1.300	1.293	10.5500	1.2965	5.2228	6.2
18	.....	.....	1.2459	0.325	0.6168	0.0	37	1.320	1.323	11.0585	1.3215	5.4745	8.2
19	.....	.....	1.5057	0.365	0.7454	1.5							

TABLE 33.—MEYER AND SEE, SHARP-CRESTED WEIRS, SET I, SERIES E.
(Date, May 16, 1914; Temperature of Water, 53 to 56° Fahr.)

No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.	No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.
	Float No. 1.	Float No. 2.						Float No. 1.	Float No. 2.				
1	.....	.....	0.0852	0.049	0.0422	16.9	20	.....	.....	1.7429	0.400	0.8624	2.4
2	.....	.....	0.1328	0.067	0.0657	13.7	21	.....	.....	1.9370	0.428	0.9584	2.6
3	.....	.....	0.1445	0.074	0.0715	6.7	22	.....	.....	1.9801	0.433	0.9798	3.3
4	.....	.....	0.2198	0.092	0.1088	17.1	23	.....	.....	2.4345	0.497	1.2046	3.2
5	.....	.....	0.2925	0.112	0.1447	16.0	24	.....	.....	2.8590	0.550	1.4146	4.1
6	.....	.....	0.3787	0.133	0.1874	16.0	25	.....	.....	3.3086	0.606	1.6371	4.2
7	.....	.....	0.4555	0.163	0.2254	2.9	26	.....	.....	3.6904	0.652	1.8260	4.2
8	.....	.....	0.5038	0.175	0.2493	2.3	27	.....	.....	4.1277	0.702	2.0424	4.3
9	.....	.....	0.5659	0.189	0.2800	2.3	28	.....	.....	4.5539	0.750	2.2533	4.2
10	.....	.....	0.6379	0.205	0.3156	2.1	29	.....	.....	4.9408	0.796	2.4445	4.5
11	.....	.....	0.7337	0.225	0.3630	2.1	30	.....	.....	5.0782	0.803	2.5102	4.8
12	.....	.....	0.7722	0.233	0.3821	2.0	31	.....	.....	5.5743	0.852	2.7582	5.3
13	.....	.....	0.8445	0.248	0.4179	1.6	32	.....	.....	6.1420	0.905	3.0391	6.0
14	.....	.....	0.9563	0.269	0.4732	1.9	33	.....	.....	6.6989	0.957	3.3146	6.3
15	.....	.....	1.0555	0.287	0.5223	2.0	34	.....	.....	7.1750	1.000	3.5502	6.6
16	.....	.....	1.1599	0.305	0.5739	2.3	35	1.101	1.103	8.2950	1.102	4.1044	6.5
17	.....	.....	1.3011	0.330	0.6438	2.0	36	1.201	1.203	9.5693	1.202	4.7349	7.9
18	.....	.....	1.4221	0.350	0.7037	2.1	37	.....	.....	10.4253	1.263	5.1585	9.1
19	.....	.....	1.5710	0.375	0.7773	1.6	38	.....	.....	11.1783	1.305	5.5311	11.4

TABLE 34.—MEYER AND SEE, SHARP-CRESTED WEIRS, SET I, SERIES F.
(Date, May 19, 1914; Temperature of Water, 58 to 59° Fahr.)

No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.	No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.
	Float No. 1.	Float No. 2.						Float No. 1.	Float No. 2.				
1	.....	.....	0.0685	0.039	0.0338	32.0	20	.....	.....	1.4917	0.358	0.7370	3.3
2	.....	.....	0.0735	0.042	0.0363	26.4	21	.....	.....	1.7787	0.400	0.8788	4.3
3	.....	.....	0.1134	0.058	0.0560	20.4	22	.....	.....	2.0862	0.443	1.0307	5.0
4	.....	.....	0.1839	0.081	0.0909	18.4	23	.....	.....	2.5365	0.504	1.2532	5.2
5	.....	.....	0.2486	0.100	0.1228	16.6	24	.....	.....	2.9884	0.559	1.4765	6.1
6	.....	.....	0.3355	0.122	0.1658	16.9	25	.....	.....	3.1026	0.573	1.5329	6.1
7	.....	.....	0.3338	0.131	0.1649	4.4	26	.....	.....	3.3245	0.597	1.6425	6.9
8	.....	.....	0.4026	0.138	0.1989	16.5	27	.....	.....	3.7884	0.647	1.8717	8.0
9	.....	.....	0.4802	0.167	0.2373	4.4	28	.....	.....	4.2083	0.699	2.0792	6.8
10	.....	.....	0.5301	0.179	0.2619	3.8	29	.....	.....	4.6733	0.747	2.3089	7.4
11	.....	.....	0.6118	0.198	0.3023	3.0	30	.....	.....	5.1835	0.802	2.5610	7.1
12	.....	.....	0.7260	0.222	0.3587	3.0	31	.....	.....	5.6770	0.848	2.8048	7.9
13	.....	.....	0.8142	0.239	0.4023	3.4	32	.....	.....	6.3048	0.904	3.1150	8.8
14	.....	.....	0.9228	0.260	0.4559	3.3	33	.....	.....	6.7761	0.949	3.3479	8.7
15	.....	.....	1.0446	0.282	0.5161	3.5	34	.....	.....	7.3858	1.001	3.6491	9.4
16	.....	.....	1.0877	0.290	0.5374	3.3	35	.....	.....	7.8850	1.045	3.8957	9.5
17	.....	....	1.1398	0.298	0.5631	3.9	36	.....	.....	8.4902	1.095	4.1948	9.9
18	.....	.....	1.2575	0.318	0.6213	4.0	37	.....	.....	9.4581	1.171	4.6730	10.7
19	.....	.....	1.3778	0.338	0.6807	4.0	38	.....	.....	11.2151	1.286	5.5411	14.1

TABLE 35.—MEYER AND SEE, SHARP-CRESTED WEIRS, SET I, SERIES G.
(Date, May 19, 1914; Temperature of Water, 59 to 62° Fahr.)

No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.	No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.
	Float No. 1.	Float No. 2.						Float No. 1.	Float No. 2.				
1	.....	.....	0.0842	0.050	0.0416	11.8	20	.....	.....	1.7562	0.396	0.8681	4.6
2	.....	.....	0.1408	0.070	0.0696	12.8	21	.....	.....	2.1561	0.448	1.0658	6.7
3	.....	.....	0.1695	0.083	0.0838	5.3	22	.....	.....	2.5174	0.495	1.2444	7.3
4	.....	.....	0.2078	0.095	0.1027	5.3	23	.....	.....	2.9800	0.552	1.4731	7.9
5	.....	.....	0.2732	0.115	0.1350	3.9	24	.....	.....	3.4317	0.602	1.6963	9.1
6	.....	.....	0.3281	0.130	0.1622	3.9	25	.....	.....	3.8702	0.651	1.9131	9.4
7	.....	.....	0.4022	0.150	0.1988	2.7	26	.....	.....	4.3106	0.697	2.1308	10.0
8	.....	.....	0.4844	0.170	0.2394	2.6	27	.....	.....	4.4302	0.709	2.1899	10.1
9	.....	.....	0.5684	0.190	0.2810	1.9	28	.....	.....	4.8624	0.755	2.4036	10.0
10	.....	.....	0.6711	0.211	0.3317	2.8	29	.....	.....	5.2641	0.795	2.6021	10.2
11	.....	.....	0.7650	0.230	0.3782	3.0	30	.....	.....	5.8713	0.850	2.9023	11.2
12	.....	.....	0.8748	0.252	0.4324	2.6	31	.....	.....	6.6473	0.919	3.2859	12.0
13	.....	.....	0.9776	0.270	0.4832	3.4	32	.....	.....	6.9851	0.946	3.4528	12.7
14	.....	.....	1.0954	0.291	0.5415	3.6	33	.....	.....	7.6575	1.004	3.7852	13.0
15	.....	.....	1.2124	0.311	0.5993	3.8	34	.....	.....	8.4390	1.065	4.1715	14.0
16	.....	.....	1.3214	0.328	0.6532	4.4	35	.....	.....	8.8501	1.097	4.3747	14.3
17	.....	.....	1.4387	0.349	0.7112	3.6	36	.....	.....	10.2516	1.200	5.0675	15.8
18	.....	.....	1.5781	0.368	0.7801	4.9	37	.....	.....	11.1942	1.249	5.5835	19.0
19	.....	.....	1.6524	0.382	0.8168	3.9							

TABLE 36.—MEYER AND SEE, SHARP-CRESTED WEIRS, SET II, SERIES B.
(Date, April 17, 1914; Temperature of Water, 40 to 47° Fahr.)

No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.	No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.
	Float No. 1.	Float No. 2.						Float No. 1.	Float No. 2.				
1	.....	.....	0.0886	0.052	0.0437								
2	0.057	0.059	0.1007	0.058	0.0497	10.3	19	0.405	0.407	1.9950	0.406	0.9847	14.2
3	.....	.....	0.1398	0.0725	0.0690	6.3	20	0.446	0.449	2.3505	0.4475	1.1602	16.2
4	0.077	0.079	0.1527	0.078	0.0754	4.0	21	0.495	0.498	2.7947	0.4965	1.3794	18.5
5	0.091	0.092	0.1979	0.0915	0.0977	6.0	22	0.514	0.517	2.9751	0.5155	1.4684	19.1
6	0.110	0.111	0.2633	0.1105	0.1300	6.1	23	0.550	0.554	3.3173	0.552	1.6374	19.8
7	0.120	0.122	0.2967	0.121	0.1464	4.4	24	0.599	0.603	3.7984	0.601	1.8748	20.9
8	.....	.....	0.3491	0.134	0.1723	5.3	25	0.647	0.651	4.3441	0.649	2.1442	23.2
9	.....	.....	0.4242	0.152	0.2094	6.0	26	0.703	0.709	4.9827	0.706	2.4594	24.4
10	.....	.....	0.4204	0.152	0.2075	5.2	27	0.739	0.744	5.4403	0.7415	2.6852	26.4
11	.....	.....	0.4980	0.170	0.2458	5.2	28	0.779	0.784	5.8853	0.7815	2.9049	26.4
12	0.199	0.200	0.6346	0.1995	0.3132	5.5	29	0.823	0.829	6.5500	0.826	3.2329	29.4
13	0.243	0.244	0.8732	0.2435	0.4310	5.7	30	0.843	0.849	6.7581	0.846	3.3357	28.8
14	0.243	0.244	0.8699	0.2435	0.4294	5.3	31	0.892	0.898	7.3347	0.895	3.6153	28.1
15	0.246	0.248	0.8915	0.247	0.4400	7.7	32	0.997	1.003	8.8876	1.000	4.3867	31.8
16	0.290	0.291	1.1493	0.2905	0.5673	8.9	33	1.060	1.067	9.9294	1.0635	4.9010	32.6
17	0.350	0.352	1.5490	0.351	0.7646	10.5	34	1.092	1.099	10.4648	1.0955	5.1652	35.2
18	0.398	0.401	1.9199	0.3995	0.9476	12.7	35	1.119	1.124	10.9522	1.1215	5.4058	36.8
							36	1.122	1.125	10.9863	1.1235	5.3792	35.7

TABLE 37.—MEYER AND SEE, SHARP-CRESTED WEIRS, SET II, SERIES C.
(Date, April 18, 1914; Temperature of Water, 48 to 52° Fahr.)

No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.	No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.
	Float No. 1.	Float No. 2.						Float No. 1.	Float No. 2.				
1	.....	.....	0.0726	0.044	0.0358	15.9							
2	.....	.....	0.1208	0.065	0.0596	7.9	20	.....	.....	2.0215	0.422	0.9978	9.5
3	.....	.....	0.1388	0.068	0.0685	15.9	21	.....	.....	2.4400	0.474	1.2043	11.0
4	.....	.....	0.1738	0.084	0.0858	5.8	22	0.505	0.506	2.6944	0.5055	1.3299	11.1
5	.....	.....	0.2348	0.103	0.1159	5.3	23	0.545	0.547	3.0764	0.546	1.5184	12.9
6	.....	.....	0.3014	0.123	0.1488	3.4	24	0.590	0.592	3.4850	0.591	1.7201	13.7
7	.....	.....	0.3772	0.142	0.1862	4.4	25	0.652	0.654	4.1167	0.653	2.0319	15.6
8	0.157	0.158	0.4405	0.1575	0.2174	4.4	26	0.699	0.701	4.6117	0.700	2.2762	16.7
9	0.179	0.181	0.5360	0.180	0.2646	4.0	27	0.738	0.739	5.0139	0.7385	2.4748	16.9
10	.....	.....	0.5422	0.181	0.2676	4.4	28	0.750	0.753	5.1776	0.7515	2.5556	17.7
11	.....	.....	0.6445	0.203	0.3181	4.5	29	0.805	0.808	5.8032	0.8065	2.8643	18.8
12	0.218	0.219	0.7196	0.2185	0.3552	4.5	30	0.851	0.853	6.3553	0.852	3.1368	19.8
13	0.239	0.240	0.8299	0.2395	0.4096	5.0	31	0.902	0.905	7.0000	0.9035	3.4551	20.7
14	.....	.....	0.9548	0.262	0.4713	5.5	32	0.930	0.933	7.3333	0.9315	3.6196	20.9
15	.....	.....	1.0623	0.280	0.5243	6.3	33	0.949	0.952	7.6059	0.9505	3.7541	21.5
16	.....	.....	1.2072	0.305	0.5958	6.3	34	1.002	1.004	8.2954	1.003	4.0944	22.4
17	0.344	0.346	1.4583	0.345	0.7198	6.6	35	1.003	1.006	8.3237	1.0045	4.1084	22.5
18	0.377	0.378	1.6890	0.3775	0.8337	8.1	36	1.096	1.098	9.6167	1.097	4.7466	24.0
19	0.420	0.422	2.0200	0.421	0.9970	9.7	37	1.151	1.153	10.4424	1.152	5.1542	25.2
							38	1.175	1.179	10.8777	1.177	5.3690	26.2

TABLE 38.—MEYER AND SEE, SHARP-CRESTED WEIRS, SET II, SERIES D.
(Date, April 25, 1914; Temperature of Water, 48° Fahr.)

No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.	No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.
	Float No. 1.	Float No. 2.						Float No. 1.	Float No. 2.				
1	.....	.....	0.0904	0.052	0.0446	12.7	20	.....	.....	1.7512	0.393	0.8644	5.5
2	.....	.....	0.1098	0.061	0.0542	7.9	21	0.416	0.417	1.9050	0.4165	0.9403	5.0
3	.....	.....	0.1432	0.073	0.0707	7.9	22	.....	.....	1.9879	0.421	0.9565	5.3
4	.....	.....	0.1768	0.085	0.0873	5.5	23	.....	.....	2.1867	0.454	1.0794	6.1
5	.....	.....	0.1982	0.091	0.0978	7.1	24	.....	.....	2.5574	0.500	1.2623	7.1
6	.....	.....	0.2729	0.114	0.1347	5.0	25	.....	.....	2.9734	0.550	1.4676	8.1
7	.....	.....	0.2982	0.121	0.1472	4.8	26	.....	.....	3.8446	0.651	1.8976	8.5
8	.....	.....	0.3448	0.134	0.1702	4.0	27	.....	.....	4.2895	0.698	2.1172	8.9
9	.....	.....	0.4387	0.158	0.2165	3.4	28	.....	.....	4.7066	0.740	2.3228	9.5
10	.....	.....	0.5191	0.177	0.2562	3.2	29	.....	.....	5.0020	0.768	2.4689	10.1
11	.....	.....	0.5737	0.190	0.2842	3.1	30	.....	.....	5.3074	0.797	2.6196	10.5
12	.....	.....	0.6782	0.212	0.3347	3.1	31	0.855	0.857	5.9427	0.856	2.9332	11.3
13	.....	.....	0.7723	0.231	0.3812	3.1	32	.....	.....	6.4963	0.905	3.2064	11.9
14	0.235	0.236	0.7992	0.2355	0.3945	3.7	33	.....	.....	7.0613	0.952	3.4853	12.7
15	.....	.....	0.8799	0.251	0.4343	3.7	34	0.996	0.997	7.5994	0.9965	3.7509	13.4
16	.....	.....	0.9927	0.272	0.4900	3.7	35	.....	.....	7.6239	1.003	3.7921	13.5
17	.....	.....	1.1862	0.305	0.5855	4.4	36	.....	.....	8.8510	1.098	4.3687	14.2
18	0.321	0.322	1.2812	0.3215	0.6324	4.2	37	1.223	1.226	10.6090	1.2245	5.2364	16.1
19	.....	.....	1.4324	0.345	0.7070	4.8	38	.....	.....	11.1553	1.259	5.5060	17.1

TABLE 39.—MEYER AND SEE, SHARP-CRESTED WEIRS, SET II, SERIES E.
(Date, May 2, 1914; Temperature of Water, 49 to 53° Fahr.)

No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.	No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.
	Float No. 1.	Float No. 2.						Float No. 1.	Float No. 2.				
1	.....	.....	0.0784	0.047	0.0387	13.5	20	.....	.....	2.0629	0.441	1.0182	4.5
2	.....	.....	0.1418	0.071	0.0700	11.3	21	.....	.....	2.1689	0.456	1.0705	4.5
3	.....	.....	0.1999	0.090	0.0987	9.7	22	.....	.....	2.5191	0.500	1.2434	5.5
4	.....	.....	0.2280	0.100	0.1125	6.9	23	.....	.....	2.9249	0.553	1.4437	5.5
5	.....	.....	0.2748	0.113	0.1356	7.3	24	.....	.....	3.2914	0.598	1.6246	5.5
6	.....	.....	0.3442	0.132	0.1699	6.3	25	.....	.....	3.7508	0.650	1.8513	6.1
7	.....	.....	0.4053	0.148	0.2000	5.5	26	0.671	0.673	3.9536	0.672	1.9514	6.3
8	.....	.....	0.4835	0.168	0.2386	4.0	27	.....	.....	4.2379	0.704	2.0917	6.3
9	.....	.....	0.5441	0.182	0.2686	3.9	28	.....	.....	4.7782	0.760	2.3584	6.9
10	.....	.....	0.5930	0.190	0.2878	4.4	29	.....	.....	5.1688	0.800	2.5512	7.1
11	.....	.....	0.6827	0.212	0.3370	3.7	30	0.850	0.851	5.6920	0.8505	2.8095	7.4
12	.....	.....	0.7670	0.230	0.3786	3.1	31	.....	.....	6.3306	0.908	3.1247	8.4
13	0.253	0.252	0.8894	0.2525	0.4390	3.9	32	.....	.....	6.7783	0.950	3.3456	8.5
14	.....	.....	0.9868	0.271	0.4871	3.9	33	1.000	1.003	7.3738	1.0015	3.6395	9.0
15	.....	.....	1.1593	0.301	0.5722	4.0	34	1.030	1.032	7.7005	1.031	3.8008	9.0
16	.....	.....	1.3469	0.333	0.6648	3.7	35	1.106	1.107	8.6109	1.1065	4.2502	9.7
17	.....	.....	1.5088	0.360	0.7447	3.6	36	1.227	1.230	10.1767	1.2285	5.0230	10.8
18	.....	.....	1.7883	0.403	0.8827	3.7	37	1.290	1.294	11.1877	1.292	5.5220	12.9
19	.....	.....	1.7950	0.403	0.8860	4.0							

TABLE 40.—MEYER AND SEE, SHARP-CRESTED WEIRS, SET II, SERIES F.
(Date, May 5, 1914; Temperature of Water, 54 to 58° Fahr.)

No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.	No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.
	Float No. 1.	Float No. 2.						Float No. 1.	Float No. 2.				
1	.....	.....	0.0716	0.046	0.0353	6.9	20	0.452	0.451	2.0938	0.4515	1.0335	2.1
2	.....	.....	0.0904	0.055	0.0446	3.4	21	0.464	0.463	2.1827	0.4635	1.0773	2.8
3	.....	.....	0.1265	0.068	0.0624	5.7	22	.....	.....	2.5050	0.506	1.2364	3.2
4	.....	.....	0.2172	0.098	0.1072	4.8	23	.....	.....	2.9005	0.557	1.4316	3.4
5	.....	.....	0.2910	0.120	0.1436	3.7	24	0.602	0.600	3.2512	0.601	1.6047	3.4
6	.....	.....	0.3595	0.141	0.1774	0.7	25	.....	.....	3.6394	0.647	1.7963	3.6
7	0.146	0.145	0.3815	0.1455	0.1883	1.6	26	0.703	0.701	4.1079	0.702	2.0276	3.4
8	.....	.....	0.4517	0.162	0.2230	2.6	27	0.706	0.705	4.1402	0.7055	2.0435	3.6
9	.....	.....	0.5226	0.180	0.2579	1.5	28	.....	.....	4.5158	0.747	2.2299	3.4
10	.....	.....	0.6159	0.201	0.3040	1.5	29	0.804	0.802	5.0433	0.803	2.4893	3.9
11	.....	.....	0.7114	0.220	0.3511	2.3	30	0.856	0.854	5.5648	0.855	2.7467	4.4
12	.....	.....	0.7998	0.239	0.3948	1.5	31	.....	.....	6.0316	0.900	2.9771	4.7
13	.....	.....	0.9275	0.263	0.4578	1.8	32	0.949	0.947	6.5477	0.948	3.2318	5.2
14	.....	.....	1.0268	0.282	0.5068	1.6	33	1.004	1.002	7.1537	1.003	3.5309	5.5
15	0.296	0.294	1.0972	0.295	0.5416	1.5	34	1.023	1.021	7.3729	1.022	3.6391	5.8
16	0.301	0.300	1.1308	0.3005	0.5581	1.6	35	1.099	1.097	8.2591	1.098	4.0766	6.5
17	.....	.....	1.3521	0.339	0.6674	1.6	36	1.200	1.198	9.4953	1.199	4.6857	7.1
18	.....	.....	1.5954	0.378	0.7875	1.8	37	1.316	1.314	11.2197	1.315	5.5379	10.3
19	.....	.....	1.8650	0.420	0.9205	1.5							

TABLE 41.—MEYER AND SEE, SHARP-CRESTED WEIRS, SET II, SERIES G.
(Date, May 7, 1914; Temperature of Water, 60 to 62° Fahr.)

No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge, in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.	No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed heads, in feet.	Discharge in second-feet, per foot of weir.	Percentage excess over discharge by Francis' formula.
	Float No. 1.	Float No. 2.						Float No. 1.	Float No. 2.				
1	.....	.....	0.0897	0.052	0.0443	11.8	20	.....	.....	1.8700	0.419	0.9230	2.1
2	.....	.....	0.1099	0.056	0.0542	22.4	21	.....	.....	2.1531	0.457	1.0627	3.1
3	.....	.....	0.1781	0.078	0.0879	21.2	22	0.500	0.498	2.4868	0.499	1.2274	4.7
4	.....	.....	0.1770	0.083	0.0874	9.7	23	.....	.....	2.6096	0.517	1.2880	4.0
5	.....	.....	0.2638	0.110	0.1302	7.3	24	.....	.....	2.8533	0.550	1.4083	3.7
6	.....	.....	0.3263	0.128	0.1611	5 5	25	.....	.....	3.2910	0.604	1.6244	3.9
7	.....	.....	0.3900	0.145	0.1925	4.7	26	.....	.....	3.6728	0.652	1.8128	3.4
8	.....	.....	0.4700	0.164	0.2320	4.8	27	.....	.....	4.1142	0.699	2.0307	4.4
9	.....	.....	0.4862	0.168	0.2400	4.7	28	.....	.....	4.6040	0.756	2.2724	3.9
10	0.181	0.180	0.5394	0.1805	0.2662	4.0	29	.....	.....	5.0631	0.803	2.4990	4.4
11	.....	.....	0.6283	0.200	0.3101	4.0	30	.....	.....	5.0982	0.807	2.5164	4.2
12	.....	.....	0.7188	0.220	0.3548	3.2	31	.....	.....	5.5274	0.851	2.7282	4.4
13	.....	.....	0.8234	0.241	0.4064	3.1	32	0.899	0.898	6.0215	0.8985	2.9721	4.8
14	.....	.....	0.9290	0.262	0.4585	2.4	33	.....	.....	6.5688	0.949	3.2422	5.3
15	.....	.....	1.0493	0.284	0.5179	2.8	34	.....	.....	7.1583	1.002	3.5332	5.8
16	.....	.....	1.1411	0.301	0.5632	2.6	35	.....	.....	8.2768	1.099	4.0853	6.6
17	.....	.....	1.2465	9.318	0.6153	2.9	36	.....	.....	9.4508	1.196	4.6647	7.1
18	.....	.....	1.3548	0.337	0.6687	2.8	37	.....	.....	9.6834	1.213	4.7795	7.4
19	.....	.....	1.6260	0.381	0.8026	2.4	38	.....	.....	11.0886	1.306	5.4731	10.1

TABLE 42.—Jones, Series A, Sharp Crest.

No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed head, in feet = h.	Average discharge, in second-feet, per foot of weir.	Percentage excess of measured discharge over discharge by Francis' formula.	No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed head, in feet = h.	Average discharge, in second-feet, per foot of weir.	Percentage excess of measured discharge over discharge by Francis' formula.
	Gauge No. 1.	Gauge No. 2.						Gauge No. 1.	Gauge No. 2.				
1	0.0198	0.0206	0.02013	0.0202	0.009944	4.34	22	0.6471	0.6478	3.5401	0.6474	1.7542	1.10
2	0.0303	0.0303	0.04102	0.0303	0.02033	15.82	23	0.7450	0.7458	4.3980	0.7458	2.1768	1.57
3	0.0400	0.0408	0.05943	0.0404	0.02945	8.90	24	0.8513	0.8528	5.3719	0.8520	2.6620	1.65
4	0.0490	0.0483	0.07717	0.0487	0.03824	6.87	25	0.9463	0.9468	6.3219	0.9465	3.1327	2.17
5	0.0595	0.0598	0.10240	0.0596	0.05074	4.73	26	1.0527	1.0565	7.4879	1.0527	3.7105	3.17
6	0.0690	0.0698	0.12772	0.0694	0.06329	3.96	27	1.1510	1.1518	8.6220	1.1514	4.2725	3.86
7	0.0800	0.0803	0.15964	0.0801	0.07911	4.80	28	1.1502	1.1518	8.6031	1.1510	4.2632	3.68
8	0.0895	0.0895	0.19098	0.0895	0.09464	6.15	29	1.3600	1.3588	11.260	1.3594	5.5798	5.72
9	0.1005	0.1003	0.22838	0.1004	0.11317	6.90	30	1.3583	1.3603	11.217	1.3583	5.5585	5.44
10	0.1185	0.1183	0.2960	0.1184	0.14668	8.11	31	0.0196	0.0196	0.01732	0.0196	0.008383	−6.07
11	0.1395	0.1388	0.3530	0.1392	0.1749	1.10	32	0.0405	0.0400	0.05718	0.0403	0.02833	5.17
12	0.1590	0.1583	0.4345	0.1587	0.2153	2.28	33	0.0808	0.0808	0.1609	0.0808	0.07973	4.26
13	0.1785	0.1783	0.5162	0.1784	0.2558	1.95	34	0.1008	0.1003	0.2258	0.1006	0.1119	5.27
14	0.1995	0.1995	0.6090	0.1995	0.3018	1.72	35	0.1193	0.1188	0.2968	0.1191	0.1471	7.45
15	0.2495	0.2498	0.8497	0.2496	0.4211	1.42	36	0.1313	0.1315	0.3302	0.1314	0.1636	3.16
16	0.2800	0.2803	1.0090	0.2801	0.5000	1.27	37	0.0308	0.0313	0.04043	0.0310	0.02003	10.24
17	0.3513	0.3513	1.4131	0.3513	0.7002	0·98	38	0.0413	0.0418	0.06085	0.0415	0.03015	7.11
18	0.3998	0.3993	1.7096	0.3996	0.8472	0.71	39	0.0528	0.0533	0.08502	0.0530	0.04213	3.69
19	0.4483	0.4483	2.0313	0.4483	1.0066	0.71	40	0.0613	0.0620	0.10643	0.0616	0.05274	3.59
20	0.4983	0.4983	2.3944	0.4983	1.1865	1.29	41	0.1388	0.1388	0.3564	0.1388	0.1766	2.43
21	0.5498	0.5508	2.7684	0.5500	1.3718	1.00	42	0.1798	0.1793	0.5195	0.1798	0.2574	1.82

TABLE 43.—Jones, Series B, Up-Stream Edge Rubbed Once Over with Fine Emery Paper Under Pressure of About 5 Pounds.

No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed head, in feet = h.	Average discharge, in second-feet, per foot of weir.	Percentage excess of measured discharge over discharge by Francis' formula.	No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed head, in feet = h.	Average discharge, in second-feet, per foot of weir.	Percentage excess of measured discharge over discharge by Francis' formula.
	Gauge No. 1.	Gauge No. 2.						Gauge No. 1.	Gauge No. 2.				
1	0.0198	0.0204	0.01998	0.0201	0.009901	4.33	17	0.3475	0.3481	1.3969	0.3478	0.6923	1.34
2	0.0288	0.0294	0.03890	0.0291	0.019276	16.60	18	0.3975	0.3986	1.7006	0.3980	0.8427	0.79
3	0.0388	0.0394	0.05642	0.0391	0.02796	8.58	19	0.4488	0.4497	2.0403	0.4492	1.0110	0.85
4	0.0488	0.0494	0.07750	0.0491	0.03840	5.99	20	0.4990	0.5039	2.4013	0.5015	1.1899	0.63
5	0.0583	0.0589	0.10018	0.0586	0.04964	5.08	21	0.5996	0.5996	3.1602	0.5996	1.5660	1.29
6	0.0683	0.0689	0.12674	0.0686	0.06267	4.75	22	0.6982	0.6990	3.9803	0.6986	1.9724	1.44
7	0.0788	0.0794	0.1578	0.0791	0.07820	5.55	23	0.7979	.....	4.8653	0.7979	2 4109	1.58
8	0.0908	0.0914	0.1962	0.0911	0.09722	6.18	24	0.9000	0.9044	5.8878	0.9022	2.9174	2.24
9	0.0983	0.0989	0.2309	0.0986	0.1095	6.21	25	0.9978	0.9979	6.8849	0.9978	3.4117	2.80
10	0.1183	0.1189	0.2875	0.1186	0.1425	4.78	26	1.1054	.....	8.1130	1.1054	4.0203	3.86
11	0.1398	0.1404	0.3637	0.1401	0.1802	3.21	27	1.1069	.....	8.1048	1.1069	4.0161	3.56
12	0.1588	0.1594	0.4390	0.1591	0.2175	2.98	28	1.1926	.....	9.1394	1.1926	4.5290	4.43
13	0.1805	0.1816	0.5296	0.1810	0.2624	2.34	29	1.1914	.....	9.0947	1.1914	4.5068	4.08
14	0.2005	0.2019	0.6175	0.2012	0.3060	1.83	30	1.3559	1.3579	11.2252	1.3569	5.5625	5.69
15	0.2478	0.2489	0.8468	0.2483	0.4196	1.85	31	1.3568	1.3579	11.2489	1.3573	5.5743	5.86
16	0.2988	0.2994	1.1154	0.2991	0.5527	1.47							

TABLE 44.—JONES, SERIES C, CREST ROUNDED 0.0065 INCH.

No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed head, in feet = h.	Average discharge, in second-feet, per foot of weir.	Percentage excess of measured discharge over discharge by Francis' formula.	No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed head, in feet = h.	Average discharge, in second-feet, per foot of weir.	Percentage excess of measured discharge over discharge by Francis' formula.
	Gauge No. 1.	Gauge No. 2.						Gauge No. 1.	Gauge No. 2.				
1	0.0196	0.0193	0.01847	0.0195	0.009153	0.94	17	0.3481	0.3478	1.3966	0.3480	0.6921	1.24
2	0.0306	0.0302	0.04107	0.0304	0.020352	15.31	18	0.4037	0.4033	1.7430	0.4035	0.8637	1.20
3	0.0401	0.0400	0.05833	0.0401	0.028903	8.13	19	0.4535	0.4525	2.0734	0.4530	1.0274	1.19
4	0.0491	0.0493	0.07783	0.0492	0.038568	6.13	20	0.4981	0.4975	2.3953	0.4978	1.1870	1.49
5	0.0606	0.0598	0.10357	0.0602	0.051324	4.34	21	0.6011	.....	3.1814	0.6011	1.5765	1.58
6	0.0706	0.0701	0.13068	0.0704	0.064758	4.12	22	0.6435	.....	3.9381	0.6435	1.9515	.....
7	0.0801	0.0798	0.15978	0.0800	0.079168	5.07	23	0.7990	0.7984	4.8940	0.7987	2.4252	2.08
8	0.0896	0.0893	0.18972	0.0895	0.094014	5.44	24	0.8992	0.8990	5.8615	0.8991	2.9046	2.31
9	0.1016	0.1018	0.23338	0.1017	0.1156	7.02	25	0.9969	0.9981	6.8750	0.9975	3.4068	2.69
10	0.1211	0.1213	0.3075	0.1212	0.1524	8.47	26	0.9966	0.9980	6.8860	0.9973	3.4123	2.89
11	0.1388	0.1383	0.3603	0.1386	0.1785	3.90	27	1.0988	1.0998	7.9891	1.0993	3.9590	3.15
12	0.1596	0.1595	0.4407	0.1596	9.2184	2.87	28	1.0982	.....	8.0249	1.0982	3.9767	3.77
13	0.1811	0.1813	0.5313	0.1812	0.2633	2.53	29	1.1941	1.1933	9.1801	1.1937	4.5490	4.74
14	0.2018	0.2021	0.6217	0.2019	0.3081	2.02	30	1.1941	.....	9.1361	1.1941	4.5273	4.27
15	0.2525	0.2581	0.8668	0.2528	0.4295	1.47	31	1.3572	1.3594	11.2161	1.3583	5.5580	5.43
16	0.3006	0.3008	1.1246	0.3007	0.5573	1.49	32	1.3578	1.3574	11.2048	1.3576	5.5524	5.42

TABLE 45.—JONES, SERIES D, CREST ROUNDED 0.0075 INCH.

No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed head, in feet = h.	Average discharge, in second-feet, per foot of weir.	Percentage excess of measured discharge over discharge by Francis' formula.	No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed head, in feet = h.	Average discharge, in second-feet, per foot of weir.	Percentage excess of measured discharge over discharge by Francis' formula.
	Gauge No. 1.	Gauge No. 2.						Gauge No. 1.	Gauge No. 2.				
1	0.0405	0.0407	0.06024	0.0406	0.02985	9.73	19	0.1605	0.1597	0.4450	0.1601	0.2205	3.37
2	0.0580	0.0582	0.10067	0.0581	0.04989	6.99	20	0.1790	0.1792	0.5250	0.1791	0.2601	3.05
3	0.0792	0.0795	0.16115	0.07935	0.07986	7.31	21	0.2002	0.2002	0.6163	0.2002	0.3054	2.38
4	0.1004	0.1008	0.2312	0.1006	0.1146	7.80	22	0.2510	0.2507	0.8600	0.2508	0.4261	1.89
5	0.1395	0.1395	0.3648	0.1395	0.1808	4.21	23	0.3000	0.2997	1.1207	0.2998	0.5553	1.59
6	0.1790	0.1784	0.5235	0.1787	0.2594	3.10	24	0.3505	0.3497	1.4067	0.3501	0.6971	1.06
7	0.2507	0.2512	0.8623	0.2509	0.4273	2.11	25	0.4020	0.4017	1.7316	0.4018	0.8581	1.17
8	0.0185	0.0182	0.01618	0.0184	0.008018	.....	26	0.4528	.....	2.0669	0.4528	1.0242	0.95
9	0.0301	0.0297	0.04052	0.0299	0.02006	16.62	27	0.4990	.....	2.4010	0.4990	1.1898	1.36
10	0.0405	0.0402	0.05973	0.0404	0.02960	9.45	28	0.6021	.....	3.1838	0.6021	1.5777	1.40
11	0.0490	0.0487	0.07793	0.0488	0.03859	7.19	29	0.6973	.....	3.9774	0.6973	1.9710	1.65
12	0.0610	0.0607	0.10068	0.0608	0.05257	5.31	30	0.7984	.....	4.8737	0.7984	2.4151	1.66
13	0.0705	0.0702	0.13200	0.0704	0.06541	5.16	31	0.9016	.....	5.8917	0.9016	2.9196	2.42
14	0.0792	0.0787	0.15917	0.0790	0.07887	6.67	32	0.9987	.....	6.9020	0.9987	3.4202	2.91
15	0.0893	0.0887	0.1920	0.0890	0.09514	7.60	33	1.1009	.....	8.0530	1.1009	3.9936	3.75
16	0.1013	0.1007	0.2330	0.1010	0.1155	8.05	34	1.1984	.....	9.1962	1.1984	4.5571	4.31
17	0.1180	0.1177	0.2870	0.1178	0.1422	5.56	35	1.3556	.....	11.2195	1.3556	5.5597	5.77
18	0.1412	0.1405	0.3707	0.1408	0.1837	4.48	36	1.3557	.....	11.2253	1.3557	5.5626	5.83

TABLE 46.—JONES, SERIES E, CREST ROUNDED 0.0120 INCH.

No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed head, in feet = h.	Average discharge, in second-feet, per foot of weir.	Percentage excess of measured discharge over discharge by Francis' formula.	No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed head, in feet = h.	Average discharge, in second-feet, per foot of weir.	Percentage excess of measured discharge over discharge by Francis' formula.
	Gauge No. 1.	Gauge No. 2.						Gauge No. 1.	Gauge No. 2.				
1	0.0209	0.0204	0.02131	0.0206	0.01056	7.26	16	0.3009	0.3004	1.1292	0.3006	0.5596	1.93
2	0.0289	0.0287	0.03893	0.0288	0.01929	18.55	17	0.3544	0.3534	1.4341	0.3539	0.7106	1.36
3	0.0389	0.0389	0.05767	0.0389	0.02858	11.85	18	0.4022	0.4017	1.7351	0.4020	0.8703	1.36
4	0.0524	0.0524	0.08578	0.0524	0.04251	6.43	19	0.4523	0.4519	2.0699	0.4521	1.0257	1.33
5	0.0694	0.0669	0.10660	0.0606	0.05283	6.34	20	0.4993	0.4997	2.3382	0.4995	1.1547	...
6	0.0694	0.0689	0.13092	0.0692	0.06488	7.02	21	0.6015	.....	3.1784	0.6015	1.5750	1.33
7	0.0799	0.0799	0.1624	0.0799	0.08048	7.00	22	0.6981	.....	3.9947	0.6981	1.9735	1.91
8	0.0909	0.0909	0.1986	0.0909	0.09841	7.80	23	0.8003	.....	4.9110	0.8003	2.4336	2.08
9	0.1036	0.1033	0.2414	0.1034	0.1196	8.03	24	0.8979	.....	5.8668	0.8979	2.9072	2.61
10	0.1176	0.1177	0.2860	0.1176	0.1417	5.51	25	1.0010	.....	6.9394	1.0010	3.4375	3.08
11	0.1406	0.1399	0.3680	0.1402	0.1824	4.35	26	1.0985	.....	8.0405	1.0985	3.9844	3.92
12	0.1624	0.1617	0.4530	0.1620	0.2245	3.41	27	1.0992	.....	8.0411	1.0992	3.9847	3.84
13	0.1812	0.1809	0.5333	0.1810	0.2643	3.08	28	1.3542	.....	11.2088	1.3542	5.5544	5.84
14	0.2016	0.2009	0.6220	0.2012	0.3092	2.56	29	1.3545	.....	11.1943	1.3545	5.5472	5.68
15	0.2514	0.2506	0.8640	0.2512	0.4281	2.12							

TABLE 47.—JONES, SERIES F, CREST ROUNDED 0.0165 INCH.

No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed head, in feet = h.	Average discharge, in second-feet, per foot of weir.	Percentage excess of measured discharge over discharge by Francis' formula.	No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed head, in feet = h.	Average discharge, in second-feet, per foot of weir.	Percentage excess of measured discharge over discharge by Francis' formula.
	Gauge No. 1.	Gauge No. 2.						Gauge No. 1.	Gauge No. 2.				
1	0.0208	0.0212	0.02090	0.0210	0.01085	2.18	16	0.3048	0.3042	1.1488	0.3045	0.5693	1.73
2	0.0291	0.0297	0.03835	0.0294	0.01950	16.15	17	0.3463	0.3472	1.3966	0.3468	0.6921	1.77
3	0.0418	0.0424	0.06453	0.0421	0.03198	11.20	18	0.4048	0.4044	1.7562	0.4046	0.8703	1.57
4	0.0523	0.0527	0.08713	0.0525	0.04318	7.79	19	0.4589	.....	2.1264	0.4589	1.0537	1.79
5	0.0608	0.0614	0.1094	0.0611	0.05421	7.79	20	0.4972	.....	2.4025	0.4972	1.1905	1.97
6	0.0710	0.0712	0.1366	0.0711	0.06769	7.22	21	0.6014	.....	3.1858	0.6014	1.5787	1.64
7	0.0813	0.0812	0.1681	0.0812	0.08330	8.13	22	0.7044	.....	4.0474	0.7044	2.0056	1.88
8	0.0948	0.0947	0.2131	0.0948	0.10560	8.64	23	0.7992	.....	4.9024	0.7992	2.4298	2.11
9	0.1050	0.1047	0.2694	0.1048	0.1335	....	24	0.8993	.....	5.8608	0.8993	2.9043	2.27
10	0.1278	0.1252	0.3257	0.1280	0.1614	5.83	25	0.9974	.....	6.8977	0.9982	3.4181	3.08
11	0.1408	0.1414	0.3740	0.1411	0.1853	4.98	26	0.9978	.....	6.8941	0.9978	3.4163	2.93
12	0.1600	0.1607	0.4487	0.1604	0.2223	3.93	27	1.0976	.....	8.0201	1.0976	3.9743	3.79
13	0.1816	0.1817	0.5383	0.1816	0.2667	3.49	28	1.3556	.....	11.1800	1.3556	5.5401	5.59
14	0.2018	0.2022	0.6283	0.2020	0.3113	2.98	29	1.3534	.....	11.2079	1.3534	5.5540	5.94
15	0.2518	0.2524	0.8708	0.2521	0.4313	2.30							

TABLE 48.—JONES, SERIES G, CREST ROUNDED 0.0415 ($\frac{1}{24}$) INCH.

No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed head, in feet = h.	Average discharge, in second-feet, per foot of weir.	Percentage excess of measured discharge over discharge by Francis' formula.	No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed head, in feet = h.	Average discharge, in second-feet, per foot of weir.	Percentage excess of measured discharge over discharge by Francis' formula.
	Gauge No. 1.	Gauge No. 2.						Gauge No. 1.	Gauge No. 2.				
1	0.0204	0.0205	0.02252	0.0204	0.01116	15.00	16	0.3032	0.3023	1.1544	0.3028	0.5720	3.08
2	0.0309	0.0310	0.04582	0.0310	0.02270	24.90	17	0.3459	.....	1.4078	0.3459	0.6974	2.95
3	0.0406	0.0407	0.06480	0.0406	0.03211	17.80	18	0.4088	0.4079	1.7984	0.4084	0.8912	2.54
4	0.0509	0.0510	0.08783	0.0510	0.04352	13.45	19	0.4548	0.4539	2.1063	0.4544	1.0438	2.33
5	0.0589	0.0585	0.1075	0.0587	0.05327	12.45	20	0.4944	.....	2.4000	0.4944	1.1887	2.69
6	0.0672	0.0675	0.1316	0.0674	0.06521	11.90	21	0.6023	.....	3.2168	0.6023	1.5938	2.39
7	0.0790	0.0790	0.1665	0.0790	0.08251	11.58	22	0.7036	.....	4.0651	0.7036	2.0144	2.50
8	0.0916	0.0917	0.2116	0.0916	0.10485	13.55	23	0.7955	.....	4.9044	0.7955	2.4308	2.86
9	0.1029	0.1030	0.2470	0.1030	0.1224	11.15	24	0.9001	.....	5.8921	0.9001	2.9198	2.68
10	0.1229	0.1225	0.3130	0.1227	0.1551	8.38	25	1.0020	.....	6.9651	1.0020	3.4515	3.34
11	0.1434	0.1430	0.3903	0.1432	0.1934	7.16	26	1.0025	.....	6.9747	1.0025	3.4562	3.40
12	0.1619	0.1615	0.4653	0.1617	0.2306	6.52	27	1.0957	.....	8.0207	1.1055	3.9746	4.07
13	0.1801	0.1792	0.5400	0.1796	0.2676	5.60	28	1.3507	.....	11.1517	1.3507	5.5261	5.70
14	0.1994	0.1993	0.6273	0.1994	0.3108	4.83	29	1.3581	.....	11.2170	1.3581	5.5585	6.05
15	0.2511	0.2513	0.8800	0.2512	0.4361	4.03							

TABLE 49.—JONES, SERIES H, SHARP CREST.

No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed head, in feet = h.	Average discharge, in second-feet, per foot of weir.	Percentage excess of measured discharge over discharge of Francis' formula.	No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed head, in feet = h.	Average discharge, in second-feet, per foot of weir.	Percentage excess of measured discharge over discharge by Francis' formula.
	Gauge No. 1.	Gauge No. 2.						Gauge No. 1.	Gauge No. 2.				
1	0.0172	0.0178	0.01502	0.0175	0.007447	.....	20	0.5021	.....	2.4126	0.5021	1.1961	0.95
2	0.0287	0.0293	0.03681	0.0290	0.01825	11.00	21	0.6014	.....	3.1713	0.6014	1.5722	1.23
3	0.0376	0.0383	0.05314	0.03808	0.02635	6.81	22	0.6011	.....	3.1657	0.6011	1.5695	1.13
4	0.0492	0.0498	0.07737	0.0495	0.03836	4.61	23	0.6995	.....	3.9824	0.6995	1.9752	1.38
5	0.0582	0.0588	0.09833	0.0585	0.04875	3.45	24	0.7000	.....	3.9825	0.7000	1.9757	1.30
6	0.0682	0.0688	0.1243	0.0685	0.06163	3.23	25	0.8007	.....	4.8867	0.8007	2.4237	1.55
7	0.0772	0.0783	0.1515	0.0778	0.07511	3.95	26	0.8014	.....	4.8951	0.8014	2.4269	1.52
8	0.0872	0.0883	0.1833	0.0878	0.09088	4.91	27	0.9020	.....	5.8583	0.9020	2.9045	1.82
9	0.1007	0.1013	0.2274	0.1010	0.1127	5.42	28	0.9001	.....	5.8510	0.9001	2.9008	2.01
10	0.1192	0.1198	0.2975	0.1195	0.1475	7.20	29	1.0007	.....	6.8843	1.0007	3.4131	2.39
11	0.1384	0.1391	0.3588	0.1388	0.1779	3.31	30	0.9998	.....	6.8802	0.9998	3.4111	2.46
12	0.1592	0.1598	0.4398	0.1595	0.2180	2.78	31	1.1103	.....	8.0091	1.1103	3.9709	2.19
13	0.1787	0.1793	0.5213	0.1790	0.2584	2.46	32	1.1111	.....	8.0260	1.1111	3.9792	2.02
14	0.1982	0.1988	0.6050	0.1985	0.2999	1.83	33	1.3552	.....	11.1315	1.3552	5.5188	5.05
15	0.2497	0.2503	0.8513	0.2500	0.4221	1.42	34	1.3567	.....	11.1287	1.3567	5.5174	4.86
16	0.3002	0.3008	1.1180	0.3005	0.5543	1.04	35	1.1123	.....	8.1513	1.1123	4.0413	3.45
17	0.3477	0.3478	1.3880	0.3477	0.6882	0.81	36	1.1120	.....	8.1646	1.1120	4.0479	3.66
18	0.4072	0.4063	1.7526	0.4068	0.8689	0.57	37	1.3564	.....	11.1772	1.3564	5.5419	5.36
19	0.4568	.....	2.0883	0.4568	1.0353	0.70	38	1.3567	.....	11.2098	1.3567	5.5576	5.62

TABLE 50.—WEBER, SERIES II, ROUGHNESS, 96 GROOVES PER INCH FOR 4 INCHES BELOW CREST LEVEL.

No. of run.	Measured discharge, in second-feet.	Mean of observed head, in feet = h.	Average discharge, in second-feet, per foot of weir.	Percentage excess of measured discharge over discharge by Francis' formula.	No. of run.	Measured discharge, in second-feet.	Mean of observed head, in feet = h.	Average discharge, in second-feet, per foot of weir.	Percentage excess of measured discharge over discharge by Francis' formula.
1	0.02122	0.0212	0.01057	2.87	20	2.3940	0.4989	1.1910	1.50
2	0.04078	0.0316	0.02032	8.62	21	3.1607	0.6006	1.5725	1.45
3	0.05703	0.0402	0.02842	5.88	22	4.0206	0.7033	2.0003	1.85
4	0.07453	0.0485	0.03714	4.42	23	4.8491	0.7952	2.4101	2.07
5	0.1023	0.0604	0.05097	5.24	24	5.7576	0.9029	2.8616	.....
6	0.1278	0.0702	0.06368	2.81	25	6.9682	1.0059	3.4633	3.09
7	0.1543	0.0788	0.07688	4.36	26	6.9623	1.0052	3.4569	3.01
8	0.1820	0.0877	0.09068	4.86	27	8.0369	1.1011	3.9905	3.71
9	0.2215	0.0997	0.1104	5.34	28	8.0279	1.0997	3.9860	3.80
10	0.2970	0.1224	0.1480	3.79	29	11.1538	1.3547	5.5381	5.48
11	0.3723	0.1430	0.1855	3.00	30	11.2054	1.3547	5.5637	5.96
12	0.4397	0.1605	0.2189	2.24	31	1.4309	0.3545	7.1242	1.37
13	0.5357	0.1835	0.2667	1.91	32	2.0564	0.4519	1.0231	1.10
14	0.6093	0.2000	0.3034	1.88	33	3.1439	0.5978	1.5626	1.53
15	0.8337	0.2470	0.4151	1.54	34	5.9942	0.9121	2.9777	2.65
16	1.1016	0.2975	0.5485	1.52	35	5.9843	0.9116	2.9728	2.58
17	1.3882	0.3480	0.6912	1.11	36	11.2082	1.3569	5.5651	5.75
18	1.7599	0.4072	0.8756	1.24	37	11.1925	1.3560	5.5573	5.69
19	2.0839	0.4572	1.0367	0.70					

TABLE 51.—WEBER, SERIES III, ROUGHNESS, 48 GROOVES PER INCH FOR 4 INCHES BELOW CREST LEVEL.

No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed head, in feet = h.	Average discharge in second-feet, per foot of weir.	Percentage excess of measured discharge over discharge by Francis' formula.	No. of run.	Observed Head, in Feet.		Measured discharge, in second-feet.	Mean of observed head, in feet = h.	Average discharge, in second-feet, per foot of weir.	Percentage excess of measured discharge over discharge by Francis' formula.
	Gauge No. 1.	Gauge No. 2.						Gauge No. 1.	Gauge No. 2.				
1	0.0234	0.0233	0.0249	0.0234	0.01243	4.28	18	0.4059	.....	1.7386	0.4059	0.8663	0.59
2	0.0301	0.0300	0.03723	0.0300	0.01859	7.46	19	0.4579	.....	2.0904	0.4579	1.0415	0.94
3	0.0389	0.0393	0.05377	0.0391	0.02684	4.24	20	0.5515	.....	2.7749	0.5515	1.3826	1.38
4	0.0484	0.0488	0.07293	0.0488	0.03641	2.04	21	0.5512	.....	2.7679	0.5512	1.3791	1.20
5	0.0584	0.0588	0.09646	0.0586	0.04816	1.95	22	0.6475	.....	3.5271	0.6475	1.7556	1.19
6	0.0689	0.0693	0.1224	0.0691	0.06111	1.025	23	0.6965	.....	3.9506	0.6965	1.9664	1.59
7	0.0782	0.0785	0.1492	0.0784	0.07449	1.92	24	0.7960	.....	4.8425	0.7960	2.4104	1.92
8	0.0889	0.0893	0.18192	0.0891	0.09081	2.54	25	0.7965	.....	4.8447	0.7965	2.4115	1.87
9	0.0979	0.0983	0.25886	0.0981	0.1293	2.64	26	0.8964	.....	5.6753	0.8964	2.8249	....
10	0.1197	0.1198	0.29130	0.1197	0.1454	5.23	27	0.8956	.....	5.7909	0.8956	2.8825	2.13
11	0.1404	0.1403	0.3580	0.1404	0.1787	2.00	28	0.9993	.....	6.8921	0.9993	3.4306	3.13
12	0.1587	0.1598	0.4357	0.1592	0.2175	2.84	29	0.9996	.....	6.8737	0.9996	3.4215	2.81
13	0.1849	0.1843	0.5381	0.1846	0.2686	1.71	30	1.0956	.....	7.9558	1.0956	3.9601	3.70
14	0.1987	0.1988	0.6000	0.1987	0.2995	1.56	31	1.0940	.....	7.9340	1.0940	3.9492	3.65
15	0.2509	0.2513	0.8503	0.2511	0.4243	1.26	32	1.3561	.....	11.1489	1.3561	5.5495	5.53
16	0.3009	0.3008	1.1144	0.3009	0.5561	1.18	33	1.3562	.....	11.1219	1.3562	5.5360	5.26
17	0.3494	.....	1.3908	0.3494	0.6940	0.89	34	1.3582	.....	11.1643	1.3582	5.5571	5.43

TABLE 52.—WEBER, SERIES IV, ROUGHNESS, 24 GROOVES PER INCH FOR 4 INCHES BELOW CREST LEVEL.

No. of run.	Measured discharge, in second-feet.	Mean of observed head, in feet = h.	Average discharge, in second-feet, per foot of weir.	Percentage excess of measured discharge over discharge by Francis' formula.	No. of run.	Measured discharge, in second-feet.	Mean of observed head, in feet = h.	Average discharge, in second-feet, per foot of weir.	Percentage excess of measured discharge over discharge by Francis' formula.
1	0.01754	0.0185	0.008759	4.52	16	1.1220	0.2997	0.5597	2.44
2	0.03557	0.0277	0.01776	15.7	17	1.4242	0.3523	0.7105	2.04
3	0.05700	0.0395	0.02846	8.88	18	1.7793	0.4082	0.8876	2.21
4	0.08087	0.0507	0.04038	6.10	19	2.0614	0.4512	1.0284	1.89
5	0.10653	0.0612	0.05320	5.22	20	2.3749	0.4950	1.1848	2.16
6	0.13420	0.0712	0.06702	5.94	21	3.1463	0.5969	1.5684	2.12
7	0.16210	0.0802	0.08095	6.57	22	4.0899	0.7093	2.0388	2.49
8	0.18904	0.0887	0.09440	7.31	23	4.8903	0.7985	2.4378	2.60
9	0.2226	0.0987	0.1116	7.65	24	5.8543	0.8991	2.9184	2.80
10	0.29167	0.1174	0.14565	8.66	25	6.9450	1.0028	3.4621	3.64
11	0.36433	0.1397	0.18175	4.59	26	8.0050	1.0945	3.9945	4.51
12	0.43367	0.1574	0.21634	3.99	27	7.9669	1.0941	3.9715	4.36
13	0.5127	0.1765	0.2558	3.60	28	11.2351	1.3565	5.6007	6.45
14	0.6083	0.1985	0.3035	3.05	29	11.2307	1.3563	5.5985	6.44
15	0.8450	0.2476	0.4215	2.71	30	11.2193	1.3548	5.5929	6.51

TABLE 53.—WEBER, SERIES V, ROUGHNESS, 24 GROOVES PER INCH FOR 4 INCHES + 15 GROOVES PER INCH FOR 8 INCHES.

No. of run.	Measured discharge, in second-feet.	Mean of observed head, in feet = h.	Average discharge, in second-feet, per foot of weir.	Percentage excess of measured discharge over discharge by Francis' formula.	No. of run.	Measured discharge, in second-feet.	Mean of observed head, in feet = h.	Average discharge, in second-feet, per foot of weir.	Percentage excess of measured discharge over discharge by Francis' formula.
1	0.01536	0.0172	0.00767	2.13	16	1.0856	0.2932	0.5423	2.59
2	0.03770	0.0294	0.01883	12.14	17	1.2583	0.3232	0.6285	2.71
3	0.05308	0.0372	0.02649	10.88	18	1.7275	0.3994	0.8628	2.65
4	0.07580	0.0484	0.03786	6.77	19	2.0184	0.4444	1.0082	2.20
5	0.09763	0.0566	0.04877	8.76	20	3.2269	0.6045	1.6118	2.98
6	0.12006	0.0654	0.05997	7.69	21	4.0857	0.7062	2.0408	3.27
7	0.14987	0.0759	0.07486	7.50	22	4.9007	0.7970	2.4479	3.27
8	0.18216	0.0864	0.09099	7.59	23	5.8314	0.8930	2.9128	3.65
9	0.22000	0.0973	0.10989	8.73	24	6.8183	0.9866	3.4057	4.37
10	0.29867	0.1182	0.1492	10.27	25	6.8001	0.9857	3.3966	4.23
11	0.37267	0.1416	0.1867	4.95	26	8.5903	1.1443	4.2968	5.27
12	0.45167	0.1616	0.2256	4.29	27	8.5807	1.1437	4.2861	5.23
13	0.52767	0.1799	0.2636	3.74	28	11.1424	1.3494	5.5656	6.02
14	0.61000	0.1986	0.3047	3.40	29	11.1935	1.3531	5.5910	6.78
15	0.83567	0.2456	0.4174	2.96					

TABLE 54.—WEBER, SERIES VI, ROUGHNESS, 24 GROOVES PER INCH FOR 4 INCHES BELOW CREST LEVEL. (DUPLICATE OF SERIES III.)

No. of run.	Measured discharge, in second-feet.	Mean of observed head, in feet = h.	Average discharge, in second-feet, per foot of weir.	Percentage excess of measured discharge over discharge by Francis' formula.	No. of run.	Measured discharge, in second-feet.	Mean of observed head, in feet = h.	Average discharge, in second-feet, per foot of weir.	Percentage excess of measured discharge over discharge by Francis' formula.
1	0.1838	0.0889	0.0918	3.96	11	2.3109	0.4913	1.1537	0.65
2	0.2804	0.1156	0.1400	6.95	12	3.0825	0.5936	1.5389	1.05
3	0.3510	0.1378	0.1752	2.88	13	3.8715	0.6876	1.9328	1.81
4	0.4427	0.1616	0.2210	2.17	14	4.7210	0.7882	2.3570	1.15
5	0.5290	0.1826	0.2641	1.61	15	5.7382	0.8923	2.8648	2.06
6	0.8293	0.2469	0.4140	1.32	16	6.8565	0.9993	3.4231	2.90
7	1.0590	0.2912	0.5287	1.05	17	6.8481	1.0014	3.4189	2.46
8	1.4256	0.3556	0.7122	0.86	18	9.0407	1.1919	4.5135	4.16
9	1.6970	0.3996	0.8472	0.71	19	11.2945	1.3622	5.6387	6.51
10	2.0125	0.4476	1.0047	0.92	20	11.2953	1.3629	5.6391	6.43

APPENDIX B

TABLE 55.—Velocities in Approach Channels (at Head Gauge Station Unless Otherwise Noted).

(Data refer to conditions approximately as for the volumetrically measured runs noted, Column (1). Head, h, determined during current-meter observations. Mean velocity, v, for entire channel obtained by volumetric measurements, $\frac{Q}{A}$, corrected to agree with head, h. Other velocities by current meter. Corrected values (Columns (5), (6), and (7)), obtained from current-meter vertical traverses divided by ratio, $\frac{v_m}{v}$ (see Figs. 33 to 47). Head, in feet; velocities, in feet per second.)

Runs No.	Heads on weir, (h).	Velocities in Approach Channel.					Runs No.	Heads on weir, (h).	Velocities in Approach Channel.				
		True mean (v).	Mean in mid-channel, vertical (v_m).	Corrected Values.					True mean (v).	Mean in mid-channel vertical (v_m).	Corrected Values.		
				At surface (v_s).	Mean above crest level (v_a).	Mean below crest level (v_b).					At surface (v_s).	Mean above crest level (v_a).	Mean below crest level, (v_b).
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(1)	(2)	(3)	(4)	(5)	(6)	(7)
Schoder and Turner, Series "A", Weir, 7.53 Feet High.													
17- 18	0.562	0.1745	0.190	0.30	0.29	0.166	39- 42	1.412	0.636	0.624	1.30	1.15	0.540
19- 23	0.760	0.269	0.272	0.56	0.50	0.245	43- 47	1.709	0.828	0.88	1.56	1.40	0.705
24- 28	0.879	0.331	0.338	0.63	0.60	0.302	48- 52	1.913	0.971	1.00	1.90	1.75	0.785
29- 33	1.000	0.402	0.429	0.75	0.70	0.364	53- 57	2.157	1.130	1.135	2.17	1.95	0.880
34- 38	1.197	0.510	0.513	1.00	0.92	0.450	59- 63	2.440	1.329	1.32	2.54	2.04	1.10
Series "B", Weir, 5.50 Feet High.													
18	0.55	0.226	0.22	0.36	0.31	0.21	45- 49	1.71	1.045	1.01	1.88	1.50	0.90
19- 23	0.76	0.347	0.35	0.61	0.54	0.32	50- 54	1.91	1.215	1.20	1.96	1.69	1.08
24- 28	0.88	0.428	0.42	0.71	0.61	0.39	55- 59	2.16	1.418	1.42	2.04	1.91	1.23
29- 33	1.00	0.518	0.54	0.80	0.73	0.48	60- 64	2.28	1.53	1.56	2.22	1.96	1.35
34- 38	1.205	0.664	0.71	1.07	0.91	0.61	65- 69	2.40	1.67	1.64	2.56	2.12	1.48
39- 44	1.416	0.826	0.89	1.27	1.12	0.75							
Series "C", Weir, 5.34 Feet High.													
76- 79	1.519	0.992	0.99	1.27	1.22	0.93	110-114	2.2626	1.63	1.48	4.13	2.89	1.05
95- 99	2.2492	1.56	1.65	2.05	1.99	1.38	115-119	2.2546	1.52	1.37	1.50	1.12	1.69
100-104	2.2296	1.885	.....	5.08*	4.71	.....	125-127	2.2350	1.875	1.95	2.88	2.75	1.47
105-109	2.2905	1.76	1.55	5.44	3.82	0.86							

*As observed; eddy at bottom.

TABLE 55.—(*Continued*).

Runs No.	Heads on weir, (h).	Velocities in Approach Channel.					Runs No.	Heads on weir, (h).	Velocities in Approach Channel.				
		True mean (v).	Mean in mid-channel, vertical (v_m).	Corrected Values.					True mean (v).	Mean in mid-channel, vertical (v_m).	Corrected Values.		
				At surface (v_s).	Mean above crest level (v_a).	Mean below crest level (v_b).					At surface (v_s).	Mean above crest level (v_a).	Mean below crest level (v_b).
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(1)	(2)	(3)	(4)	(5)	(6)	(7)
Schoder and Turner, Series "D", Weir, 7.50 Feet High.													
31- 35	1.0018	0.396	0.449	0.78	0.67	0.395	71- 75	2.2454	1.173	1.13	1.62	1.49	1.08
36- 40	1.2008	0.512	0.508	0.90	0.76	0.472	76- 80	2.2454	1.179	1.143	1.74	1.57	1.06
41- 45	1.3981	0.635	0.634	1.00	0.85	0.596	81- 85	2.2518	1.178	1.160	1.44	1.46	1.10
46- 50	1.6029	0.769	0.764	1.26	1.04	0.710	86- 90	2.2501	1.168	1.205	1.74	1.21	1.15
51- 55	1.8052	0.889	0.885	1.59	1.30	0.789	91- 95	3.2478	1.141	1.105	0.78	0.63	1.80
56- 60	1.9995	1.020	1.05	1.84	1.76	0.898	96-100	2.2491	1.235	1.309	3.32	2.88	0.75
61- 65	2.2565	1.227	1.35	2.25	2.08	0.973	101-105	2.5100	1.433	1.58	2.60	2.36	1.125
66- 70	2.2555	1.165	1.10	1.40	1.22	1.155	106-111	2.7612	1.592	1.680	2.63	2.40	1.30
Series "L", Weir, 7.50 Feet High.													
23- 27	0.9014	0.340	0.394	0.59	0.54	0.316	33- 37	2.2268	1.210	1.383	2.36	1.96	0.99
28- 32	1.6032	0.761	0.768	1.49	1.20	0.66							
Series "M", Weir, 7.50 Feet High.													
28- 32	0.9984	0.394	0.409	0.87	0.76	0.345	53- 57	2.0008	1.032	0.927	2.42	2.10	0.75
33- 37	1.1994	0.510	0.542	1.14	0.99	0.43	58- 62	2.2140	1.181	1.135	2.50	2.08	0.92
38- 42	1.4076	0.638	0.636	1.43	1.19	0.537	63- 67	2.4084	1.325	1.293	2.72	2.33	1.00
43- 47	1.5989	0.762	0.782	1.68	1.39	0.628	68- 72	2.5318	1.412	1.341	3.07	2.61	0.96
48- 52	1.8005	0.892	0.878	2.03	1.67	0.70							
Series "K", Weir, 5.50 Feet High.													
25- 27	0.8024	0.380	0.337	0.46	0.48	0.368	61- 65	1.9984	1.260	1.05	2.13	1.36	1.22
28- 30	0.8986	0.444	0.443	0.56	0.57	0.425	66- 70	2.0014	1.255	1.17	1.19	0.66	1.47
31- 35	0.9995	0.514	0.514	0.69	0.65	0.49	71- 75	1.9972	1.390	1.33	4.08	3.07	0.77
36- 40	1.2054	0.661	0.614	1.01	0.92	0.60	76- 80	1.9927	1.365	1.29	3.26	2.62	0.91
41- 45	1.3976	0.811	0.762	1.29	1.08	0.74	81- 85	1.9819	1.326	1.49	2.45	1.98	1.09
46- 50	1.6009	0.976	0.912	1.58	1.24	0.89	86- 90	2.1968	1.49	1.504	2.32	2.03	1.28
51- 55	1.8135	1.147	1.128	1.63	1.44	1.04	91- 95	2.4140	1.683	1.733	2.53	2.26	1.44
56- 60	2.0111	1.314	1.30	1.98	1.72	1.17	96-100	2.6564	1.898	1.848	2.94	2.70	1.50
Series "O", Weir, 5.50 Feet High.													
25- 29	1.0022	0.517	0.508	0.78	0.72	0.48	50- 54	2.0019	1.320	1.272	2.49	1.91	1.11
30- 34	1.2017	0.662	0.604	1.20	1.03	0.58	55- 59	2.1997	1.500	1.449	2.77	2.20	1.22
35- 39	1.4000	0.816	0.793	1.56	1.33	0.71	60- 64	2.4200	1.700	1.750	2.98	2.53	1.34
40- 44	1.6034	0.980	0.938	1.87	1.44	0.85	65- 69	2.5591	1.830	1.750	3.24	2.70	1.43
45- 49	1.8106	1.151	1.080	2.27	1.71	0.92	70- 74	2.6901	1.962	2.12	3.50	2.98	1.47

TABLE 55.—(*Continued*).

Runs No.	Heads on weir, (*h*).	Velocities in Approach Channel.					Runs No.	Heads on weir, (*h*).	Velocities in Approach Channel.				
		True mean (*v*).	Mean in mid-channel vertical (v_m).	Corrected Values.					True mean (*v*).	Mean in mid-channel vertical (v_m).	Corrected Values.		
				At surface (v_s).	Mean above crest level (v_a).	Mean below crest level (v_b).					At surface (v_s).	Mean above crest level (v_a).	Mean below crest level (v_b).
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(1)	(2)	(3)	(4)	(5)	(6)	(7)
Schoder and Turner, Series "J", Weir, 4.0 Feet High.													
23- 25	0.8002	0.495	0.486	0.56	0.56	0.484	59- 63	1.9948	1.593	1.520	2.29	1.36	1.706
26- 28	0.9016	0.584	0.559	0.68	0.68	0.566	64- 68	2.0032	1.591	1.580	1.21	0.85	1.96
29- 33	1.0002	0.672	0.649	0.78	0.77	0.647	69- 73	1.9950	1.756	1.606	5.10	2.98	1.12
34- 38	1.2015	0.856	0.819	1.12	0.97	0.820	74- 78	2.0019	1.695	1.631	3.48	2.45	1.11
39- 43	1.3998	1.046	1.009	1.42	1.15	1.008	79- 83	2.2090	1.870	1.843	2.57	2.25	1.695
41- 48	1.6026	1.244	1.141	1.76	1.43	1.164	84- 88	2.4069	2.088	2.12	2.72	2.55	1.80
49- 53	1.7991	1.444	1.348	2.01	1.68	1.34	89- 93	2.7012	2.397	2.345	3.17	3.00	1.98
54- 58	2.0027	1.655	1.614	2.29	1.93	1.516							
Series "I", Weir 3.0 Feet High.													
23- 25	0.8002	0.630	0.612	0.70	0.66	0.623	59- 63	1.9958	1.948	2.065	1.78	1.13	2.49
26- 28	0.9041	0.742	0.716	0.83	0.78	0.728	64- 68	2.0055	1.976	1.86	2.54	1.85	2.07
29- 33	1.0006	0.846	0.835	0.96	0.91	0.827	69- 73	2.0047	2.217	2.11	4.62	3.58	1.85
34- 38	1.2031	1.070	1.026	1.30	1.15	1.06	74- 78	2.0024	2.104	1.87	4.09	3.11	1.43
39- 43	1.3986	1.293	1.238	1.58	1.35	1.265	79- 83	2.1999	2.260	2.185	2.89	2.57	2.04
44- 48	1.6042	1.531	1.465	1.92	1.68	1.46	84- 88	2.4128	2.517	2.445	3.24	2.90	2.20
49- 53	1.7955	1.762	1.761	2.40	2.08	1.60	89- 93	2.6708	2.842	2.810	3.77	3.36	2.38
54- 58	2.0079	2.024	2.03	2.60	2.33	1.82							
Series "P", Weir 2.50 Feet High.													
1- 3	1.005	0.965	0.91	1.44	1.02	0.943	7- 9	2.025	2.240	2.24	2.66	2.33	2.16
4- 6	1.013	0.994	0.81	2.27	1.68	0.72	10-12	2.022	2.355	2.27	4.55	2.76	2.01
Series "H", Weir, 2.0 Feet High.													
22-24	0.8018	0.872	0.844	0.98	0.93	0.852	53-57	1.5982	2.015	1.889	2.45	2.08	1.966
25-27	0.9016	1.012	0.964	1.27	1.06	0.933	58-62	1.8063	2.331	2.235	2.75	2.37	2.30
28-32	1.0027	1.155	1.095	1.44	1.20	1.134	63-67	2.0100	2.637	2.585	3.19	2.75	2.52
33-37	1.0020	1.145	1.040	1.22	0.98	1.229	68-72	1.9971	2.558	2.545	2.04	1.48	3.64
38-42	1.0023	1.180	1.152	2.10	1.63	0.958	73-77	2.0034	2.967	2.475	6.60	4.00	1.92
43-47	1.2026	1.435	1.344	1.68	1.48	1.40	78-82	2.1989	2.975	3.165	3.53	3.24	2.68
48-52	1.3998	1.721	1.640	2.00	1.69	1.75	83-87	2.4810	3.363	3.63	3.74	3.67	2.98
Series "G", Weir, 1.50 Feet High.													
23- 24	0.5002	0.604	0.611	0.71	0.63	0.598	56- 60	1.6017	2.430	2.342	2.84	2.45	2.43
25- 27	0.8024	1.083	1.096	1.19	1.11	1.087	61- 65	1.8036	2.777	2.684	3.80	2.82	2.72
28- 30	0.9084	1.257	1.211	1.42	1.25	1.26	66- 70	1.9938	3.114	3.026	4.57	3.25	3.04
31- 35	1.0035	1.418	1.376	1.61	1.43	1.414	71- 75	2.0004	3.387	3.510	4.34	4.42	2.03
36- 40	0.9986	1.492	1.346	3.11	2.30	0.955	76- 80	1.9827	3.146	3.280	0.72	1.92	4.78
41- 45	1.0018	1.415	1.240	1.64	1.07	1.650	81- 85	2.2026	3.490	3.357	4.40	3.59	3.34
46- 50	1.2072	1.760	1.705	2.02	1.77	1.755	86- 90	2.4041	3.845	3.825	4.90	4.04	3.845
51- 55	1.4065	2.097	2.027	2.43	2.14	2.05							

TABLE 55.—(*Continued*).

Runs No.	Heads on weir, (h).	Velocities in Approach Channel.					Runs No.	Heads on weir, (h).	Velocities in Approach Channel.				
		True mean (v).	Mean in mid-channel vertical (v_m).	Corrected Values.					True mean (v).	Mean in mid-channel vertical (v_m).	Corrected Values.		
				At surface (v_s).	Mean above crest level (v_a).	Mean below crest level (v_b).					At surface (v_s).	Mean above crest level (v_a).	Mean below crest level (v_b).
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(1)	(2)	(3)	(4)	(5)	(6)	(7)
SCHODER AND TURNER. SERIES "F", WEIR, 1.00 FEET HIGH.													
20	0.3998	0.620	0.597	0.73	0.69	0.591	52- 56	1.3994	2.660	2.67	3.10	2.73	2.55
22- 23	0.5008	0.818	0.810	0.97	0.87	0.782	57- 61	1.6026	3.06	3.18	3.46	3.09	2.80
24- 25	0.7013	1.225	1.238	1.15	1.86	1.137	62- 66	1.8031	3.50	3.53	4.18	3.67	3.21
26- 28	0.7990	1.428	1.51	1.76	1.50	1.367	67- 71	2.0055	3.90	3.99	4.38	4.11	3.48
29- 31	0.9052	1.643	1.67	2.02	1.78	1.526	72- 76	2.0040	4.07	3.73	6.32	4.44	3.33
32- 36	1.0034	1.845	1.88	2.18	1.92	1.775	77- 81	1.9950	3.99	4.72	1.94	3.56	4.84
37- 41	1.0018	1.890	1.875	2.84	2.19	1.598	82- 86	2.2122	4.28	4.44	4.78	4.58	3.77
42- 46	1.0027	1.845	2.03	1.09	1.49	2.204	87- 91*	2.3616	4.72	4.59	5.35	5.05	3.97
47- 51	1.2042	2.255	2.25	2.64	2.35	2.156							
SERIES "N", WEIR, 0.75 FEET HIGH.													
21	0.2022	0.324	0.318	0.35	0.34	0.319	51- 55	0.9888	2.218	2.265	0.91	1.64	2.90
22	0.2537	0.437	0.448	0.47	0.46	0.429	56- 60	1.2088	2.64	2.62	2.77	2.72	2.51
23	0.2988	0.537	0.533	0.58	0.57	0.524	61- 65	1.4063	3.11	3.17	3.37	3.20	2.91
24	0.3485	0.650	0.674	0.77	0.70	0.627	66- 70	1.6037	3.55	3.615	3.60	3.65	3.39
25	0.4020	0.770	0.803	0.89	0.82	0.744	71- 75	1.8187	3.99	4.21	3.51	4.04	3.88
26- 29	0.4532	0.892	0.928	1.00	0.93	0.868	76- 80	2.0016	4.43	4.675	4.27	4.55	4.12
30- 31	0.5008	0.996	0.988	1.19	1.05	0.963	81- 85	2.0092	4.63	4.93	4.53	4.77	4.36
32- 34	0.7034	1.478	1.560	1.70	1.56	1.41	86- 90	2.0168	4.36	4.35	3.81	4.22	4.71
35- 37	0.8026	1.710	1.825	2.03	1.79	1.62	91- 95	2.2045	4.86	5.05	4.49	4.96	4.55
38- 40	0.9032	1.950	2.025	2.24	2.05	1.85	(Reset)	2.1907	4.82	5.01	.....	4.89	4.63
41- 45	1.0025	2.178	2.28	2.44	2.27	2.04	96-100	2.3326	5.21	5.38	4.47	5.35	4.77
46- 50	1.0001	2.202	2.616	2.41	2.28	2.10							
SERIES "E", WEIR, 0.50 FEET HIGH.													
16	0.2088	0.472	0.568	0.52	0.49	0.467	33- 37	1.2756	3.345	3.182	3.97	3.55	2.88
18	0.3018	0.733	0.825	0.80	0.75	0.723	38- 42	1.4109	3.956	3.815	4.53	4.08	3.57
20	0.4020	1.020	1.090	1.31	1.13	0.932	43- 47	1.5912	4.460	4.380	5.04	4.55	4.18
22- 24	0.7990	2.158	2.02	2.68	2.36	1.860	48- 52	1.8099	5.120	5.300	5.65	5.25	4.62
25- 27	0.9000	2.482	2.365	3.14	2.63	2.05	53- 57	2.0004	5.775	6.083	6.80	5.98	4.95
28- 32	1.0028	2.777	2.684	3.41	3.02	2.30							
MEYER AND SEE, SET I, SERIES C, WEIR, 2.068 FEET HIGH.													
29	0.803	0.869	0.87	0.92	0.92	0.92	37	1.330	1.616	1.58	1.94	1.82	1.48
33	1.001	1.135	1.165	1.16	1.17	1.12							
SET I, SERIES E, WEIR, 1.409 FEET HIGH.													
30	0.818	1.160	1.137	1.04	1.16	1.16	38	1.306	2.038	2.09	1.66	2.18	1.90
34	1.004	1.480	1.52	1.36	1.52	1.46							

*Original table gives data also for verticals, 0.5 ft. from either wall.

TABLE 55.—(*Continued*).

Runs No.	Heads on weir, (h).	Velocities in Approach Channel.					Runs No.	Heads on weir, (h).	Velocities in Approach Channel.				
		True mean (v).	Mean in mid-channel vertical (v_m).	Corrected Values.					True mean (v).	Mean in mid-channel vertical (v_m).	Corrected Values.		
				At surface (v_s).	Mean above crest level (v_a).	Mean below crest level (v_b).					At surface (v_s).	Mean above crest level (v_a).	Mean below crest level (v_b).
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(1)	(2)	(3)	(4)	(5)	(6)	(7)
MEYER AND SEE, SET I, SERIES G, WEIR, 0.755 FEET HIGH.													
29	0.800	1.689	1.83	1.43	1.55	1.83	37	1.248	2.759	2.90	2.60	2.68	2.89
33	1.002	2.146	2.39	1.48	1.90	2.46							
J. O. JONES, SERIES "A", WEIR, 2.50 FEET HIGH AND 2.018 FEET LONG IN ALL SERIES.													
18	0.401	0.294	.....	0.30	0.294	0.293	29, 30	1.3613	1.447	.....	1.79	1.71	1.31
24	0.8612	0.808	.....	0.61	0.85	0.787							
SERIES "C".													
18	0.3910	0.283	.....	0.27	0.288	0.283	31, 32	1.3560	1.437	.....	1.68	1.63	1.340
24	0.8825	0.833	.....	0.85	0.828	0.839							
SERIES "D".													
26	0.440	0.332	.....	0.29	0.283	0.341	35, 36	1.3553	1.442	.....	1.85	1.71	1.30
31, 32	0.9419	0.906	.....	0.91	0.905	0.907							
SERIES "E".													
19	0.456	0.351	.....	0.28	0.289	0.363	28, 29	1.3581	1.446	.....	1.77	1.68	1.32
24, 25	0.9471	0.915	.....	0.89	0.89	0.925							
SERIES "F".													
19	0.462	0.360	.....	0.31	0.318	0.37	28, 29	1.3562	1.440	.....	1.74	1.67	1.32
24- 26	0.9299	0.889	.....	0.86	0.87	0.895							
SERIES "G".													
18	0.395	0.291	.....	0.29	0.287	0.291	28, 29	1.3592	1.450	.....	1.82	1.71	1.31
24- 26	0.9242	0.884	.....	0.95	0.909	0.875							
SERIES "H".													
21, 22	0.572	0.469	.....	0.43	0.422	0.475	37- 38	1.3567	1.440	.....	1.80	1.72	1.29
27- 30	0.956	0.921	.....	0.92	0.926	0.917							

TABLE 55.—(*Continued*).

Runs No.	Heads on weir, (h).	Velocities in Approach Channel.					Runs No.	Heads on weir, (h).	Velocities in Approach Channel.				
		True mean (v).	Mean in mid-channel vertical (v_m).	Corrected Values.					True mean (v).	Mean in mid-channel vertical (v_m).	Corrected Values.		
				At surface (v_s).	Mean above crest level (v_a).	Mean below crest level (v_b).					At surface (v_s).	Mean above crest level (v_a).	Mean below crest level. (v_b).
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(1)	(2)	(3)	(4)	(5)	(6)	(7)
C. M. Weber, Series II, Weir, 2.50 Feet High and 2.018 Feet Long in all Series.													
19	0.450	0.343	0.365	0.25	0.283	0.355	36, 37	1.363	1.450	1.473	1.81	1.628	1.353
34–35	0.935	0.899	0.923	0.86	0.853	0.915							
Series III.													
17	0.349	0.243	0.264	0.25	0.250	0.242	33, 34	1.357	1.439	1.445	1.86	1.625	1.341
27–29	0.979	0.950	0.948	0.94	0.932	0.950							
Series IV.													
19, 20	0.471	0.373	0.396	0.35	0.346	0.378	28–30	1.356	1.450	1.480	1.83	1.635	1.350
24, 25	0.942	0.913	0.882	0.85	0.881	0.927							
Series V.													
19	0.5196	0.439	0.393	0.41	0.422	0.473	28, 29	1.352	1.448	1.470	1.72	1.604	1.365
24, 25	0.9666	0.952	0.948	1.005	0.989	0.939							
Series VI.													
10, 11	0.470	0.363	0.355	0.36	0.357	0.364	19, 20	1.362	1.459	1.456	1.60	1.574	1.396
14, 15	0.854	0.797	0.802	0.67	0.689	0.832							

APPENDIX C

Velocity of Approach

This topic has been considered by many hydraulic experimenters. Seemingly, each generation in turn has studied it anew, has placed the thought on record, and forthwith has abandoned it again to the realm of speculation.

A few interesting quotations from hydraulic literature follow:

Hamilton Smith states:*

"The question to be solved is, what portion of the kinetic energy, due to the head, $\frac{v^2}{2g}$, * * * produces a useful effect at the opening of discharge [*i. e.*, above the crest-level]. * * * This subject has engaged the attention of many great minds during the past two centuries, and the discussions and controversies, if collected, would fill several good-sized volumes. If the same amount of thought and time, devoted to these almost fruitless discussions, had been employed in ascertaining what the *facts* really were, we would know a good deal more about the effect of velocity of approach than we do at present."

* * * * * * * *

"It seems reasonable to suppose that the additional force produced by the velocity of approach is chiefly due to the velocity of the section of the feeding stream nearest or in line with the section of discharge where the particles of water fairly begin to form into the escaping vein. This velocity is unquestionably greater than v (the mean velocity)."

Francis states:†

"It not infrequently happens that, in consequence of the particular form of the canal leading to the weir, or from other causes, the velocity of the water in the canal is not uniform in all parts of the section; this is a frequent cause of serious error, and is often entirely overlooked. If great irregularities exist they should be removed by causing the water to pass through one or more gratings, presenting numerous small apertures equally distributed, or otherwise, as the case may require, through which the water may pass under a small head; these gratings should be placed as far from the weir as practicable."

Francis, in deriving his theoretical expression involving velocity of approach, assumes a uniform velocity throughout the cross-section of the channel of approach. This condition is, so to speak, not a natural one, as had been recognized early in the Nineteenth Century by Lesbros, Boileau, and others.

Bazin states:‡

"* * * in varying the height of a weir, we modify not only the mean velocity, v, of approach, but also the distribution of the velocities in the water above the weir, the thickness of the falling sheet, its contraction on the sill, etc."

Bazin, however, apparently made no measurements to ascertain the extent of (or the degree of constancy in the law of) the modifications in the distribution of velocities for different heights of weir.

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* "Hydraulics" (1886), pp. 68–69, 84.

† "Lowell Hydraulic Experiments" (1871), p. 135.

‡ *Annales des Ponts et Chaussées*, October, 1888; *Proceedings*, Engineers' Club of Philadelphia, January, 1890, pp. 307–308.

TABLE OF CONTENTS

	PAGE
SYNOPSIS	999
EXPERIMENTAL EQUIPMENT:	
The Experimental Canal	1004
Water Supply	1004
Baffles	1005
"Fences"	1006
Weir Bulkhead	1006
Weirs of Series A to P:	
Series A	1006
Series B	1006
Dawson, Series I	1006
Dawson, Series II	1007
Dawson, Series III	1007
Series C	1007
Series D to P	1007
METHOD OF EXPERIMENTATION BY WRITERS:	
General Method	1008
Head Measurements:	
Float-Gauge and Piezometers	1008
Plumb-Bob Gauge	1011
Zero Determinations	1012
Discharge Measurements:	
Diverter	1013
Hoisting Tackle	1013
Wasteway	1014
Stand-Pipe Measuring Tank	1015
Stand-Pipe Float-Gauge	1015
Time Measurements	1015
Procedure in a Typical Run	1016
Primary Calculations from Observed Data:	
Leakage Corrections	1017
Velocity Measurements:	
Velocities	1020
Adjustment of Velocities	1021
NEW EXPERIMENTAL DATA BY OTHERS:	
Walker and Weidner's Experiments	1022
Martin's Experiments	1022
Dawson's Experiments	1023
Meyer and See's Experiments	1023
Jones' Experiments	1023
Weber's Experiments	1025
DISCUSSION ON THE NEW DATA:	
General	1025
New Data	1032
Old Formulas Tested by New Measurements:	
Fteley and Stearns' Formula	1035
Bazin's Coefficients and Formula	1035
Lyman's Averaging Weir Curves	1035
Rehbock's Formula	1035
Comments	1036

PAGE

Search for a Comprehensive Formula:
Low Weirs with High Heads..... 1037
Low Heads on Sharp-Crested Weirs..... 1042
Effects on Discharge of Slight Roundings of the Crest Corner:
Jones' Data..... 1050
Bazin's Results 1051
Effects on Discharge of Degree of Roughness of Up-Stream Face of Crest Piece:
James Barr's Experiments..... 1052
Weber's Data 1052
Bazin's Discharges Too High for Sharp Crest and Smooth Plate Condition 1053
Review of Studies:
Early Comparisons by the Writers:
Six Weirs in Same Channel..... 1054
Old and New Data..... 1065
Effect of Velocity of Approach..... 1065
General Comparison of All Old and New Data..... 1068

CONCLUSIONS INDICATED:
Condition of Crest Piece..... 1068
Velocity Distribution in the Channel of Approach..... 1068
Measurements of Head..... 1069
Friction in the Channel of Approach..... 1069

ACKNOWLEDGMENTS 1069

APPENDIX A:
Tables 9 to 54.....1071–1101

APPENDIX B:
Table 55: Velocities in Approach Channels.....1102–1107

APPENDIX C:
Velocity of Approach: Quotations from the Classics..... 1108

DISCUSSION

H. W. KING,* M. AM. SOC. C. E. (by letter).—These experiments explain possible causes of discrepancy in the results of earlier investigators and show quite clearly some of the conditions that affect the discharge over weirs. The most important of these conditions appear to be: (1) Sharpness of crest; (2) degree of roughness of face of weir; and (3) distribution of velocities in channel of approach. Although it has been known for some years that all these factors affect the discharge over thin-edged weirs their importance had not before been pointed out so clearly.

The new experiments indicate that opinions regarding the accuracy of weir measurements must be modified and also that the operations of measurement are more complicated than has heretofore been supposed. It still appears that the weir is a useful device for measuring water where a probable error of 2 to 5% is permissible. The statement of the authors that:

"The use of such a weir properly built, of good head measurements and good auxiliary velocity measurements in the channel of approach, should insure discharge values within 2% of the truth, and in most cases within 1 per cent."

is undoubtedly a fair conclusion. The standard requirements of a sharp crest and smooth up-stream face, however, are not easy to obtain and maintain and the additional requirement of measuring velocities in the channel of approach takes much from the usefulness of what has heretofore been considered a simple means of measuring water.

The new experimental data consistently give smaller discharges than were obtained by Bazin, and thus, in general, conform to the results of Francis, Fteley and Stearns, and Rehbock. The evidence appears to be overwhelmingly in favor of the smaller values. In this connection, however, it should be noted that Nagler's experiments,† which were performed on a weir, duplicating Bazin's standard weir, agreed substantially with the results obtained by Bazin. Although the discharges were not measured volumetrically, they were measured in a manner and under conditions such that their accuracy can hardly be doubted. It seems more likely that in duplicating approximately the conditions of the two sets of experiments the discharges obtained in each case were substantially the same.

In the discussion‡ of Professor Nagler's paper, the late Frederic P. Stearns, Past-President, Am. Soc. C. E., makes the following statement:

"In view of these large differences in the results of carefully conducted weir experiments, two conclusions seem to be warranted:

(1) That in the present state of the art the weir is not an accurate instrument for the measurement of water;
(2) That the art should be developed so that the causes of the variation in results may be discovered and eliminated."

* Prof. of Hydr. Eng., Univ. of Michigan, Ann Arbor, Mich.

† "Verification of the Bazin Weir Formula by Hydro-Chemical Gaugings," by Floyd A. Nagler, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 105.

‡ *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 172.

The first of these conclusions holds to-day. The later experiments only raise the query as to whether it is worth while to follow the suggestion contained in Mr. Stearns' second conclusion, or whether it will be more profitable in the end to let the sharp-crested weir rest temporarily and take up the investigation of other more promising means of measuring flowing water.

There is undoubtedly great need for some device of the weir or "obstruction" type for measuring discharges in open channels. The question arises as to whether this need is best met by the sharp-crested weir, or whether some other form of weir or obstruction in the channel can be designed for which the discharge will not be so greatly affected by minute variations in standard conditions. This is a field worthy of thorough investigation. In the past, many engineers appear to have accepted the sharp-crested weir as a panacea, and it has been investigated to the exclusion of other devices of a similar character.

The Society's Special Committee on Irrigation Hydraulics is investigating methods of measuring water in open channels, and is making a special study of devices for producing flow at critical depth. No recommendations as to a standard design have yet been made. It appears, however, from the work of the Committee to date* that discharges can be measured with great accuracy by a proper utilization of the critical-depth principle.

The discharge over a broad-crested weir occurs at critical depth. If the up-stream edge of such a weir, Fig. 50(*a*), and the up-stream edges of the ends of the weir are well rounded so as to prevent contraction, the discharge will be given very nearly by the formula,

$$Q = 3.087\ L\ H^{\frac{3}{2}}$$

This formula has the inherent objection, in common with formulas for sharp-crested weirs, that the head, H, must be corrected for velocity of approach and that this correction is affected by the distribution of velocities in the channel of approach. It has advantages, however, over the sharp-crested weir in not requiring a sharp crest which is difficult to maintain and in being much less sensitive to conditions affecting roughness of crest.

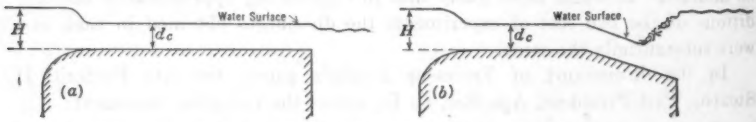

FIG. 50.—TYPES OF BROAD CRESTED WEIRS.

If the weir crest, Fig. 50(*a*), has a slope a little greater than is required to overcome the loss of head due to friction for discharge at critical depth, the water should flow at very nearly the critical depth, d_c, over the entire surface of the weir. Near the up-stream edge of the weir, just below the place where the drop in water surface occurs, the depth should be the critical depth

* "The Improved Venturi Flume," by Ralph L. Parshall, Assoc. M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 89 (1926), p. 841; particularly discussions by Julian Hinds and J. C. Stevens, Members, Am. Soc. C. E., pp. 859 and 872.

within the limits in accuracy of measurement. The discharge is then given by the theoretical formula,

$$Q = L \sqrt{g}\, d_c^{\frac{3}{2}}$$

This formula is independent of conditions affecting velocity of approach and appears to have many advantages. It still lacks a thorough experimental test.

The broad-crested weir has a distinct advantage in that the discharge is not affected when the water surface on the down-stream side is a distance, d_c, or less, above the weir crest. A still greater submergence is permissible if the down-stream portion of the crest is given a steep slope, Fig. 50(*b*). This permits the water to obtain a velocity greater than the velocity at critical depth, and a hydraulic jump results. The loss of head in a low jump is comparatively small, and by proper design most of the energy of the water can be converted into static head. The utilization of the critical-depth principle gives promise of providing a simple, accurate, and comparatively inexpensive method of measuring water.

TABLE 56.—GAUGE READINGS, IN FEET, SHOWING ANGLE OF PRESSURE ABOVE WEIR.

Gauge No. 1.	Gauge No. 2.	Gauge No. 3.	Gauge No. 4.	Gauge No. 5.	Gauge No. 6.	Gauge No. 7.	Gauge No. 8.	Gauge No. 9.
0.145	0.144	0.145	0.145	0.145	0.144	0.144	0.144	0.145
0.301	0.299	0.301	0.299	0.300	0.300	0.300	0.300	0.300
0.436	0.433	0.436	0.434	0.435	0.435	0.436	0.434	0.436
0.620	0.614	0.619	0.613	0.618	0.617	0.618	0.617	0.618
1.007	0.990	1.005	0.988	1.004	1.000	1.004	1.001	1.002
1.316	1.286	1.311	1.291	1.315	1.308	1.312	1.309	1.310
1.371	1.342	1.364	1.327	1.366	1.357	1.363	1.358	1.361
1.443	1.406	1.437	1.412	1.441	1.431	1.437	1.433	1.435
1.608	1.573	1.603	1.568	1.606	1.593	1.602	1.596	1.599
1.692	1.658	1.680	1.643	1.686	1.671	1.680	1.675	1.677
1.806	1.763	1.797	1.754	1.801	1.784	1.794	1.788	1.791
2.058	2.001	2.051	1.998	2.053	2.031	2.047	2.038	2.042
2.239	2.180	2.235	2.179	2.234	2.206	2.229	2.217	2.223
2.438	2.382	2.438	2.379	2.434	2.403	2.431	2.416	2.426
2.586	2.532	2.581	2.503	2.580	2.543	2.573	2.555	2.566
2.769	2.693	2.757	2.641	2.754	2.711	2.744	2.724	2.738
2.989	2.908	2.980	2.878	2.980	2.932	2.960	2.943	2.954
3.240	3.144	3.224	3.111	3.222	3.165	3.197	3.177	3.192
3.507	3.403	3.508	3.386	3.497	3.430	3.475	3.448	3.464
3.709	3.626	3.695	3.581	3.706	3.629	3.697	3.648	3.664

An unpublished thesis, by C. J. McLean, Assoc. M. Am. Soc. C. E., submitted in connection with graduate work at the University of Michigan, contains the results of a series of experiments for determining the variation in head measured at different places in the channel leading up to a weir. A summary of Mr. McLean's experiments is contained in Table 56. These experiments supplement similar experiments by Fteley and Stearns,* and show clearly the existence of the "angle of pressure".

The experiments were performed in the experimental flume at the University of Michigan, which is fully described in Professor Nagler's paper. Heads were measured with hook-gauges that gave a range in reading of 4 ft. Open-

* *Transactions*, Am. Soc. C. E., Vol. XII (1883), p. 1, and Vol. LXXXIII (1919–20), p. 174.

ings were made in the face of the weir and on one side of the channel, at places selected for measuring heads. In these openings short pipes, 2 in. in diameter, were placed so as to be flush with the inner surface of the channel. The pipes were connected by a $\frac{3}{4}$-in. rubber hose to 5-gal. galvanized-iron pails. The pails were raised and lowered to positions that were convenient for

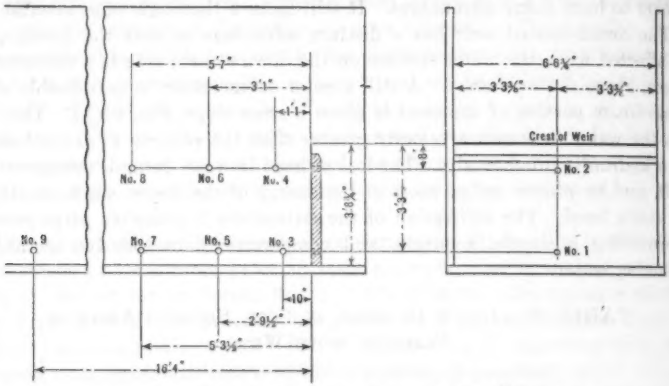

FIG. 51.—LOCATION OF CONNECTIONS TO HOOK-GAUGES.

reading the gauges. Nine connections were made at the positions indicated in Fig. 51. The location of the connection for Gauge No. 9 corresponds to that used by Bazin. Gauges Nos. 7 and 8 are connected at points corresponding approximately to points used, respectively, by Francis and Fteley and Stearns.

FLOYD A. NAGLER,* M. AM. SOC. C. E. (by letter).—The outstanding contribution to the knowledge of the hydraulics of weirs made by this excellent paper exists in the investigation of the effect of irregular channel velocities on the quantity of water flowing over the crest. Some of the divergence between the results of standard experiments could no doubt be attributed to such a cause if the precise knowledge of the velocity conditions in the approach channel were available. Certainly, far different conditions existed in the long unobstructed channel leading to the Bazin weir than in the short, wide, and turbulent channel approaching the Fteley and Stearns 19-ft. weir, the actual current velocities of which were only partly described by the results of the current-meter measurements recorded with their experiments. The writer believes that future experimenters will use extra caution in securing a satisfactory distribution of flow in the approach channel. This distribution of flow will be examined by preliminary velocity measurements, and the discharge will be determined by some standard weir formula rather than by methods allowing a poor distribution of flow to exist, which would require the measurement of current velocities for every observed head on the weir in order to solve for discharge in the authors' formula.

The experiments cited by the authors on the effect of rounding the weir crest and roughening the weir plate are also important, and are in good

* Prof. of Hydr. Eng., Univ. of Iowa, Iowa City, Iowa.

agreement with investigations which have been conducted since 1923 at the hydraulic laboratory of the State University of Iowa.

The Iowa investigations were performed on a sharp-crested weir, 2 ft. long and 3 ft. high. The crest of the weir plate was made of brass with a sharp and well maintained edge. The procedure followed by James Barr was used in roughening the weir plate by coating it with sand of several degrees of fineness. Care was exercised to file the crest after applying each coat so as to leave an irregular but sharply defined up-stream edge on the weir. How effective the roughening near the crest was in comparison with that some distance below it, was tested by stripping the roughened coat from the plate to various distances below the edge of the plate.

The appearance of the weir plate as prepared for several of the experiments is shown in Figs. 52 and 53. The weir in Fig. 52 was covered with coarse sand that was retained on a No. 8 standard Tyler sieve, but passed the No. 4 sieve. The weir in Fig. 53 is studded with hardware.

While these experiments were performed primarily to determine the effect of the roughening on the profile of and velocities within the nappe, volumetric measurements were made to compare the discharge of the weir with its flow when its crest was sharp and the weir plate smooth. A summary of the results of these measurements are given in Table 57.

It is interesting to note that the roughening of the weir plate near the crest was much more effective in increasing the discharge than the roughening more than ½ in. away, and that roughening the wooden bulkhead by heavy hardware caused no alteration in discharge.

TABLE 57.—PERCENTAGE OF INCREASE IN DISCHARGE DUE TO ROUGHENING OF WEIR PLATE.

Series No.	Condition of weir plate.	HEAD ON WEIR.			
		0.2 ft.	0.3 ft.	0.4 ft.	0.5 ft.
	Coarse sand:				
2	(Retained on No. 8 standard sieve and passing No. 4) to crest	6.5	6.0	6.0	4.7
3	To crest with bulkhead studded	6.5	6.0	6.0	4.7
4	To ¼ in. from crest	0.2	1.0	2.3	2.9
	Medium sand:				
5	(Retained on No. 28 sieve and passing No. 14) to crest	4.0	2.4	2.0	1.5
6	To 1/16 in. from crest	1.2	1.5	1.5	1.5
7	To ⅛ in. from crest	0.7	0.7	0.6	1.5
9	To ½ in. from crest	0.3	0.3	0.0	0.0

Experiments performed by the writer* have demonstrated that the Bazin formula may be used in computing discharges for heads even higher than those included in the writer's experiments, securing results much nearer the truth than those obtained by the solution of the Francis formula. The authors' experiments seem to substantiate this conclusion. Under low heads the results obtained with the Bazin formula are higher than those obtained by most other experimenters, but the explanation of the authors seem somewhat presumptuous and misleading with regard to the true condition of the

* *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 105.

Bazin weir. After considerable study and investigation the writer has come to the conclusion that the Bazin weir plate was unquestionably sharp and smooth, and for the following reasons:

(1) Hydraulic science has never known a more careful and better trained observer than M. Bazin, and it appears almost unthinkable that a man who had spent a large part of his life carefully studying into every small detail of the phenomena of the nappe passing over the standard weir, should have carelessly allowed the weir plate to become rough and the edge other than sharp. When Bazin used the words "arête vive",* an expression was used such as would correctly be applied to the sharp and well-defined knife-edge of a fine chemical balance, and it has been correctly applied to the weir in translating it as a sharp edge. Also, the words, "bien dressee"† applied to the weir plate indicate that considerable attention was given to the condition of the face of the weir plate itself.

(2) A few years ago the writer raised the question of the condition of the Bazin weir plate in some correspondence with M. Hegly, who it will be recalled was associated with Bazin in his weir experiments.‡ M. Hegly made reply in no uncertain terms, which the writer has translated as follows:

"No special photographs were taken of the crest of the standard weir used in the Bazin experiments. I only know he gave much attention to the question of assuring himself that the edge, where the under surface of the nappe escaped, was level and corresponded to a straight line. And, several times, the crest, that is to say, the plate of wrought iron 7 mm. thick, which rested on top of the wooden bulkhead, was removed and laid on the side of the experimental canal, in order that he might by touch as well as by sight determine the sharpness of the up-stream edge of the plate. He never permitted any doubt on this question.

"From time to time he cleaned the top and up-stream face of the weir plate in order to remove small deposits of algæ and rust which might have affected the accuracy of the crest."

These statements of M. Hegly leave no doubt as to the condition of the Bazin weir crest. In fact, Bazin was probably more meticulous in maintaining a sharp edge on the weir crest than any of the other early investigators.

(3) Bazin's measurements on the profile of the lower face of the weir nappe unknowingly demonstrate both the sharpness of the crest and the smoothness of the weir plate.

Measurements of the weir nappe made in the hydraulic laboratory of the State University of Iowa, have shown that the value of ϵ (Fig. 54) varies not only with the height of the weir, but with both the degree of sharpness of the crest and also the roughness of the up-stream face of the weir. A summary of the results of these experiments is given in Table 58. The weir was 3 ft. high and 2 ft. long.

Bazin obtained an average value of ϵ equal to 0.112 on a weir 3.72 ft. high, and an average value of 0.099 on a weir 1.15 ft. high.§ Arithmetical interpolation between the values for the Bazin weirs gives a value for ϵ of

* *Annales des Ponts et Chaussées*, October, 1888, pp. 395, 400, and 444.
† *Loc. cit.*, p. 400.
‡ *Loc. cit.*, p. 448.
§ *Loc. cit.*, 1890, p. 14.

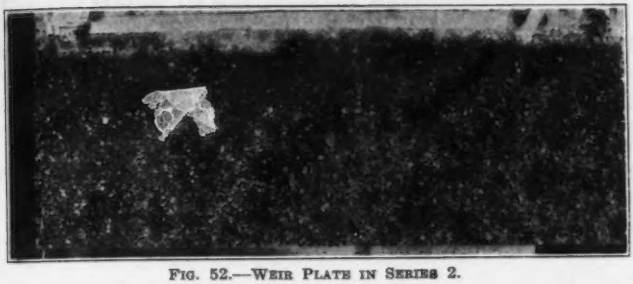

FIG. 52.—WEIR PLATE IN SERIES 2.

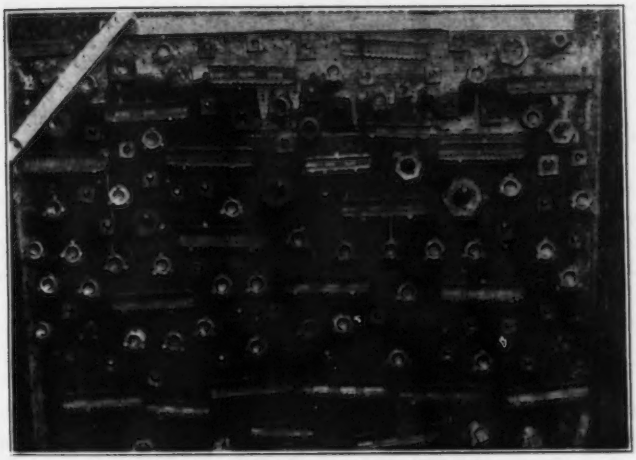

FIG. 53.—WEIR BULKHEAD FOR SERIES 3.

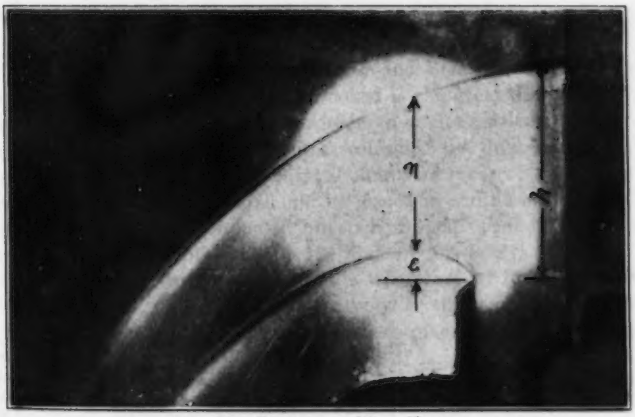

FIG. 54.—PROFILE OF WEIR NAPPE.

0.108 for a weir 3 ft. high, similar to that of the writer, but with a crest condition duplicating that of Bazin. This value is exactly that obtained on the weir at the University of Iowa, which the writer can affirm was thoroughly sharp.

TABLE 58.—MAXIMUM ELEVATION OF LOWER SURFACE OF WEIR NAPPE.

Series.	Condition of weir.	Value of $\frac{e}{h}$.
1	Standard sharp crest	0.108
5	Sand to crest	0.092
6	Sand to 1/16 in. from crest	0.097
7	Sand to 1/8 in. from crest	0.099
8	Sand to 1/4 in. from crest	0.102
9	Sand to 1/2 in. from crest	0.103

These facts make it appear conclusive that the Bazin weir had a truly sharp edge, and it is regretted that the authors' ingenious explanation of the divergence of the Bazin low-head experiments seems untenable.

The type of formula proposed by S. M. Woodward, M. Am. Soc. C. E.,* has many practical advantages in solution, which make it far more usable than that proposed by the authors; and if this formula had been checked against the original data instead of against the authors' Equation (*D*), it probably would be found to represent these data with equal accuracy.

For work in preparing the tables in this discussion, acknowledgment is due to Messrs. J. W. Howe,† E. T. Schuleen,‡ and Glen N. Cox,‡ Graduate Students at the State University of Iowa.

A. STREIFF,§ M. AM. SOC. C. E. (by letter).—Ever since Trautwine in 1889 published a translation of Bazin's experiments, Bazin's formula has been widely used in this country, and hence experiments tending to prove its somewhat large results are of importance.

As Professor Schoder so conclusively shows, the distribution of the velocity of approach is of vital importance in a weir, and it is, therefore, not easy to determine whether Bazin's or Francis, Fteley and Stearns, or Rehbock's formula is the correct one. Not to be overlooked is the great difficulty involved in calibrating a large measuring basin properly, such as a lock with more or less rough walls, as used by Francis, or a section of the Sudbury conduits, as used by Fteley and Stearns. It is easy to obtain an error of 1 or 2 per cent. Professor Rehbock's experiments should be placed in a different class. These are "indoor" experiments, conducted on such a small scale that this latter circumstance may easily introduce results that are 1% or 2% too small. The writer is well acquainted with the hydraulic laboratory of Professor Rehbock, and believes that the measurements were very accurate; but the small size of the flume no doubt caused the experiments to give somewhat small results, if the formula was applied to the measurement of larger discharges.

* See p. 1040.

† Now Assoc. M. Am. Soc. C. E.

‡ Now Jun. Am. Soc. C. E.

§ Cons. Engr., Jackson, Mich.

It is of interest to quote the experiments made at the Amsteg Power House of the Swiss Federal Railways.* The results are given in Table 59.

Seemingly, Table 59 would show the greater accuracy of the Rehbock formula over those of Frese or Bazin, especially since the volumetric measurement is closely checked by the titration method and the current meter. Nevertheless, the table shows the tendency of the Rehbock formula to be on the small side, and the Swiss "S. I. A." ("Schweizerische Ingenieur und Architekten Verein") formula is shown to check more closely than that of Rehbock.

TABLE 59.—RESULTS OF EXPERIMENTS AT AMSTEG POWER HOUSE.

Discharge, in volumetric liters per second.	PERCENTAGE ERROR OF WEIR.		
	Frese.	Bazin.	Rehbock.
1 182	+ 1.7	+ 1.5	+ 0.2
2 128	+ 1.9	+ 1.4	− 0.1
3 204	+ 2.5	+ 1.9	+ 0.2
4 371	+ 1.5	+ 0.7	− 0.8

More comparable with the Francis, Fteley and Stearns, and Schoder experiments than the Rehbock formula are the Frese experiments. In Hanover, Germany, Professor Frese (like Francis) made "full-sized" tests in a lock chamber,† which are practically unknown in this country, and are, for instance, not mentioned by R. E. Horton, M. Am. Soc. C. E., in the well-known *Water Supply Paper No. 200.* It happens that the Frese formula also gives results tending to be on the large side, and, hence, excluding Rehbock's formula, there are also the following formulas based on large-scale experiments: Francis, Fteley and Stearns, Schoder, on the small side, and Bazin and Frese, giving larger discharges.

It is of importance that Professor Schoder has demonstrated so conclusively that the weir is a very sensitive contrivance. On the Continent of Europe the inherent instability of the weir has been known and appreciated for a long time. Dr. J. Epper, former Director of the Hydrometric Survey in Switzerland, in correspondence with the writer, in 1914, stated:

"Here we do not use weirs unless they are first calibrated by the current meter. It appears that, no matter how closely the standard weir is copied, differences will creep in due to differences in the distribution of flow above the weir."

In a work published by that Survey, Dr. Epper quotes‡ the water supply of the City of Berne where weirs gave different results due to varying flow.

For measurements that are not continuous the obvious conclusion from Professor Schoder's experiments is, therefore, to discard the weir and use the current meter instead. If the weir must be supplemented by meter measure-

* "Essais comparatifs dans les canaux de fuite de l'usine d'Amsteg," Département fédéral de l'intérieur, Berne, 1926.

† "Versuche über den Abfluss des Wassers bei vollkommenen Ueberfallen," *Zeitschrift* des Vereines Deutscher Ingenieure, 1890.

‡ "Die Entwickelung der Hydrometrie in der Schweiz," Berne, 1907.

ments, the use of the meter only is obviously indicated. The weir naturally is more practical in cases where a long series of measurements is to be made. A weir arranged for continuous observation is often found, therefore, which is calibrated beforehand by other means, such as the moving screen. At the hydro-electric plant at Ackersand, Switzerland, and at the testing flume of the J. M. Voith Company, in Heidenheim, Germany,* such a combination of weir and moving screen may be found.

As a rule, however, careful observation of standard weir specifications, especially if the velocity of approach is low or, in other words, if the weir is high, will result in good measurements that are reliable. In Table 60 the writer gives the results of hitherto unpublished tests made by William G. Fargo, M. Am. Soc. C. E., and a comparison of relative turbine discharges, as measured by current meters and Francis weir, at the Loud Development on the Au Sable River, in Michigan, run on October 16 and 17, 1913. They are presented by courtesy of William W. Tefft, M. Am. Soc. C. E., Vice-President of the Consumers Power Company, present owners of the dam (1927). The weir was of the standard Francis type, probably the largest ever built. It was 117.583 ft. long; the average weir crest elevation was 216.040 ft. The concrete bottom was at Elevation 207.47, the height of the weir, therefore, being about 8.5 ft. It was solidly constructed of a steel frame set in a concrete foundation and covered with wooden gates having a steel-plate face. The weir crest was made in strict accordance with Francis' specifications. The leakage was determined by lowering the tail-race with the penstock gates closed, allowing the leakage through the gates to fill up the penstocks. It amounted to not more than 5 cu. ft. per sec. The head was measured in six places, 21 ft. 9 in. to 23 ft. 6 in. apart, by means of pipe connections placed in concrete pedestals on the bottom of the tail-race. These communicated with six barrels at which the head was measured by six hook-gauges. An aeration pipe was placed on the down-stream side. The flow was measured simultaneously by current meters just up stream from the penstock head-gates.

Two sections were selected, one being measured by Price meters, the other with an Ott meter. The meters had been carefully calibrated in the Naval Tank at the University of Michigan by Professor Herbert C. Sadler. Since the tests of S. Hajós, of the Hungarian Hydrographic Service, published in 1901,† there can be little doubt that a still-water rating is fully equivalent to one in ordinary flowing water, without cross-currents or eddies. Although these tests were made many years ago, they were not widely published and hence the same question has been re-opened many times.‡ The theory and behavior under conditions of oblique flow; the influence of different mountings; and the size of the measuring section of the Ott meters used, had been extensively investigated, and the writer, therefore, could recommend, with

* "Vergleichsversuche mit Flügel und Schirmapparat zur Bestimmung von Wassermengen," O. Lutschg, Berne, 1913; "Die Turbinenversuchsanstalten der Firma," J. M. Voith. *Zeitschrift* des Vereines Deutscher Ingenieure, 1909.

† "Beiträge zur Frage über die Umlaufswerte Woltmannscher Flügel," S. Hajos, Budapest, 1901.

‡ "Characteristics of Cup and Screw Current Meters," by B. F. Groat, M. Am. Soc. C. E. *Transactions*, Am. Soc. C. E., Vol. LXXVI (1913), p. 819.

TABLE 60.—COMPARISON OF RELATIVE TURBINE DISCHARGES.

Kilowatt output as per switchboard instruments.	Unit No.	Run No.	Revolutions per Minute:			Gate:		Time, Inclusive.			Temperature, in Degrees Fahrenheit.		Remarks.	
			Maximum.	Minimum.	Average.	Inches open. Full gate, 5¼ in.	Opening, in percentage.	Date (1913).	Begin.	End.	Atmosphere.	Water.		
			(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
2 100	2	1	122.6	117.3	120.4	3 25/32	75	Oct. 16	8 : 52	3 : 58	51	51	North-east wind, cloudy.	North wind, rain.
2 138	2	2	121.4	119.1	120.5	3 25/32	75	Oct. 16	4 : 17	5 : 28	47	51	North wind.	Rain.
1 364	2	3	121.4	118.8	120.3	2 9/16	55	Oct. 17	7 : 07	11 : 39	50	51	North-east wind.	Rain.

NOTES.—The switchboard instruments give results 4½ to 6½% too low.
In Column (43) using h = head in Column (33) and adding leakage per Column (41).
" " (54) an observation at each point took 50 sec.
" " (58) width × depth − ½ × 12 sq. in. = area = 26 × depth − ½ × 12 sq. ft.
" " (63) an observation required approximately 8 periods of 25 rev. each.
" " (67) (width + 6 × ½ in. − 2 × 6½ in.) depth = (26 + 6 × ½ in. − 2 × 6½ in.) depth.
Total observing time for one Epper-Ott meter = 14 455 sec.
" " " " five Price meters = 15 000 ".
Run No. 2 was an experiment on aeration on Francis weir.
Slide-rule computations generally.

NOTES ON WEIR.—Weir crest = Elevation 216.04004 = average of 27 readings { Maximum = 216.041. Minimum = 216.398.
Weir length = 117.583.
Average bottom at 28 ft. up stream = Elevation 205.28, at weir = Elevation 207.47.
Gamma = 62.3.

TABLE 60.—(*Continued.*)

Kilowatt output as per switchboard instruments.	Unit No.	Run No.	Switchboard Instruments.				Tail-water on wheels.	Variation in tail-water on wheels. Column (17) —Column (36).	Head Water.				
			Alternator.		Exciter.				Loss.		At gauging station.		
			Amperes.	Volts.	Amperes.	Volts.			Pond No. 27.	Pond to gauging station.	Average.	Maximum.	Minimum.
			(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)	(23)
2 100	2	1	488.5	2 575	176.5	86.4	218.12	—0.04	242.76	0.16	242.60	242.62	242.58
2 138	2	2	468.8	2 744	194.5	93.8	218.15	—0.03	242.77	0.16	242.61	242.62	242.60
1 364	2	3	374.7	2 544	141.1	70.0	217.71	—0.00	243.01	0.08	242.93	242.94	242.92

Kilowatt output as per switchboard instruments.	Unit No.	Run No.	Head, Q_5.		Vacuum Head on Weir, in Inches.		Tail-water on weir.	Head on Francis Weir.					Head-water on weir.
			H, on turbines.	$\frac{Q_5}{4\sqrt{H}}$ per runner.	No. 35, left.	No. 33, right.		Hook No. 1, right.	Indicating weir gauge No 2, ⅕ point.	Hook No. 3, No. 4, No. 5, ⅖. ⅗. ⅘ points.	Hook No. 6, left.	Average.	
			(24)	(25)	(26)	(27)	(28)	(29)	(30)	(31)	(32)	(33)	(34)
2 100	2	1	24.48	61.7	0	0	213.9	2.119	2.116	2.132	2.115	2.120	218.160
2 138	2	2	24.46	62.6	0	0	213.8	2.135	2.130	2.136	2.150	2.138	218.178
1 364	2	3	25.22	42.6	0	0	213.7	1.674	1.666	1.672	1.684	1.674	217.714

TABLE 60.—(*Continued.*)

Kilowatt output as per switchboard instruments.	Unit No.	Run No.	Francis Weir Discharge per Foot.					Total weir discharge, $L = 117.583$ Q_1.	Leakage, Q_3, through weir.	Effective water through unit under test = Q_5.
			Q_2.	Area of channel of approach.	Velocity of approach.	Percentage of increase in Q_2.	Final Q_2'.			
			(35)	(36)	(37)	(38)	(39)	(40)	(41)	(42)
2 100	2	1	10.279	12.88	0.8	0.7	10.351	1 217	5	1 222
2 138	2	2	10.410	12.90	0.8	0.7	10.483	1 233	5	1 238
1 364	2	3	7.212	12.43	0.6	0.5	7.248	852	5	857

Kilowatt output as per switchboard instruments.	Unit No.	Run No.	Comparison of Francis Weir Discharges by Various Formulas.								
			In Cubic Feet per Second.					As a Percentage of Francis.			
			Francis.	Braschman.	Bazin.	Frese.	Average.	Francis.	Braschman.	Bazin.	Frese.
			(43)	(44)	(45)	(46)	(47)	(48)	(49)	(50)	(51)
2 100	2	1	1 222	1 235	1 224	1 233	1 228	100	101.1	100.2	101.0
2 138	2	2	.....	.....	.....	.....	.....	....	.....	.....	.....
1 364	2	3	857	869	859	859	861	100	101.4	100.2	101.2

TABLE 60.—(*Continued*)

Kilowatt output as per switchboard instruments.	Unit No.	Run No.	Five Small Price Meters.						
			Treatment at Gauging Section.			Area of Gauging Section, in Square Miles.			
			Number of different verticals. (52)	Number of different points. (53)	Number of vertical curves. (54)	With stop-log grooves. (55)	Without stop-log grooves. (56)	Without stop-log grooves and meter rack. (57)	Final net area used. (58)
2 100	2	1	18	785	50	610	587	575	581
2 138	2	2	....	....	....	....	....	....	....
1 364	2	3	18	725	45	618	595	583	589

Kilowatt output as per switchboard instruments.	Unit No.	Run No.	Epper-Ott Meter.								
			Velocity.	Flow.	Treatment at Gauging Section.			Area of Gauging Section, in Square Miles.			
			Average as per curves. (59)	Final as per Columns (46) and (47). (60)	Number of different verticals. (61)	Number of different points. (62)	Number of vertical curves. (63)	With stop-log grooves and beams. (64)	Without stop-log grooves. (65)	Without stop-log grooves and beams. (66)	Final net area used. (67)
2 100	2	1	2.38	1 382	12	100	12	517	498	477	482
2 138	2	2	....	.....	....	....	....	....	....	....	....
1 364	2	3	1.63	960	9	12	9	526	506	485	490

TABLE 60.—(*Continued.*)

Kilowatt output as per switchboard instruments.	Unit No.	Run No.	Final Comparison.				
			Velocity.	Flow.	Current Meters.		Meters and Weir.
			Average as per curve.	Final as per Columns (69) and (70).	Price meter as a percentage of Epper-Ott meter.	Epper-Ott meter as a percentage of Price meter.	Percentage of Francis Discharge.
			(68)	(69)	(70)	(71)	Price. (72)	Epper-Ott. (73)
2 100	2	1	2.54	1 227	112.6	88.7	113.1	100.4
2 138	2	2	1.76	...864	111.1	90.0	112.0	100.8
1 364	2	3						

full confidence, a comparison test of the Ott meter with the weir.* The rating curves for the Ott meter had been computed by Dr. Epper, of Berne, Switzerland, who wrote that the Naval Tank, at Ann Arbor, apparently was very accurate, as shown by the small probable error. The results of the test which was conducted with great care, are shown in Table 60. It appears that the Price meters gave discharges much too great, but the agreement between the weir and Ott meter was very satisfactory. The present Ott meter (Type V), is a great improvement over that used in these tests, and gives a rating curve that is practically a straight line from the start.

After it had thus been proved conclusively that the Ott meter is fully equivalent to the weir, no more testing weirs were built, and the measurements were conducted with Ott meters exclusively. In order to make such measurements quickly and accurately, the writer introduced the use of the Tchebycheff formula, as mentioned by Dr. Staus in his recent work on the accuracy of current-meter measurements.†

R. L. PARSHALL,‡ ASSOC. M. AM. SOC. C. E. (by letter).—The measuring weir has been one of the most popular subjects investigated by hydraulic engineers.

Flowing water is not always what it appears to be from the standpoint of consistency of motion. Engineers are apt to make certain assumptions as to the line of travel and rate of displacement of the filament in the water mass, that conditions do not fully justify. All parts of a mass of water moving under the influence of gravity do not advance uniformly. For instance, that portion touching the confining surfaces is slowed down, while at some point within the mass the rate of motion may be either accelerated or retarded. This unequal movement disturbs the flow and sets up cross-currents which result in boils or eddies. This disturbed condition of flow becomes less apparent, however, with slower velocities, and at an extremely slow rate of translation the effects of the cross-currents are inappreciable, although they are present. When the stream is led over a weir notch, the velocity of the water is then sufficient to produce unequal variations of movement in the approach section, causing a departure from the ideal. The contracting vein introduces other complications which, in combination, set up a condition far from the ideal conception of stream lines curved gracefully over the crest and reaching through into the nappe of the falling sheet of water.

Flowing water is endowed with still another very troublesome characteristic. The water surface is considered fixed and constant as to elevation or depth, when in reality this is not its condition. An oscillating pulse is set up when the mass of water begins to move in an open channel. This changes

* "Hydrometrische Versuche," J. W. Sandstrom, Stockholm, 1912; "Expériences et Théories sur le moulinet de Woltmann," A. Rateau, *Annales des Mines*, 1898; "Theorie und Konstantenbestimmung des hydrometrischen Flügels," A. Ott, Berlin, 1925; "Die königliche Versuchsanstalt für Wasser und Schiffbau in Berlin," Eger, Dix und Seifert, 1908; "Der normal und schiefgestellte hydrometrische Flügel," Vienna, 1903; "Einfluss der Wandungen auf die Umlaufwerte hydrometrischer Flügel," Vienna, 1899; "Versuche über die Genauigkeit der Woltmannschen Flügel bei schräger Strömung," Frese, *Zeitschrift* des Vereines Deutscher Ingenieure, 1886; and "Untersuchungen über die Umlaufbewegung hydrometrischer Flügel," M. Schmidt, Münich, 1903.

† "Ueber den Genauigkeitsgrad von Flügelmessungen," A. Staus, Berlin, 1926.

‡ Irrig. Engr., U. S. Dept. of Agriculture, Fort Collins, Colo.

the depth or pressure and causes an unsteadiness of movement over the crest of a weir. Screens, baffles, barriers, skimmers, and all known mechanical means of damping this effect do not totally eliminate the incessant rise and fall of the surface. Experience seems to indicate that under laboratory conditions and refinement of control, certain combinations of flow conditions very greatly affect the accuracy or dependability of measurement, and for this reason exactness in weir measurement is extremely difficult to attain, if attainable at all.

The authors' experience, as indicated by the various diagrams, is that all observations do not plot along the ideal curve, and the writer's experience with weirs under laboratory conditions shows the same difficulty. It might even be stated that it is doubtful if there is any such thing as precise weir measurements.

The writer is not in full accord with the authors' method of determining the effective head on the weir crest to the limit of accuracy stated. Two methods are described. In one, a sharp-pointed plumb-bob is attached to a steel tape graduated to hundredths of a foot; in the other a float-gauge is used. The least known graduation marked on either tape or gauge is 0.01; by estimation the nearest 0.001 part is determined, and the nearest 0.0001 part is recorded as a mean. In the original notes (Test 55) the observations of heads are shown where the variation is 0.013 ft. for the plumb-bob determination, while the float-gauge readings varied between limits of 0.023 ft. The attempt to determine the mean head to 0.0001 ft. for this range of fluctuation appears to be going beyond precision, as the word should be understood in such work. However, it is to be noted in this case that the agreement of the means of the two independent measurements is remarkably close. As shown in the original notes (Test 14), both float and plumb-bob gauge readings are recorded to the nearest 0.0005 ft., which calls for the estimation of one-twentieth of the 0.01 division of the scale. This seems to claim an accuracy not to be credited.

Examination of the recorded data shows that the variation or trend of the law of discharge is not perfectly uniform. The variations are not due to inaccurate measurement of the physical constants, but to the inherent characteristics of the flowing water. In Table 42 are given results of discharge over a sharp-crested weir for heads of less than 0.02 ft., and a corresponding discharge, in second-feet, to a limit of 0.00001, with a direct comparison to a computed discharge by means of Francis' formula. There seems to be no logical reason in setting up a comparison of observed and computed free-flow discharge over a sharp-crested weir for a head of only 0.02 ft. The writer's laboratory experience with weirs indicates that about 0.07 ft. for a rising head is required to prevent adherence to the downstream face of the weir crest. A stream which adheres to the crest certainly can not follow the same law called for by the Francis formula. Perhaps it might be said with equal propriety that there is no justification for setting up comparisons of discharges, thus determined, with those computed by the Francis formula, where the heads exceed one-third the crest length.

An extensive series of tests on various types of weirs was conducted at the Hydraulic Laboratory of the Colorado Agricultural Experiment Station, at Fort Collins, for the purpose of extending the law of flow over weirs

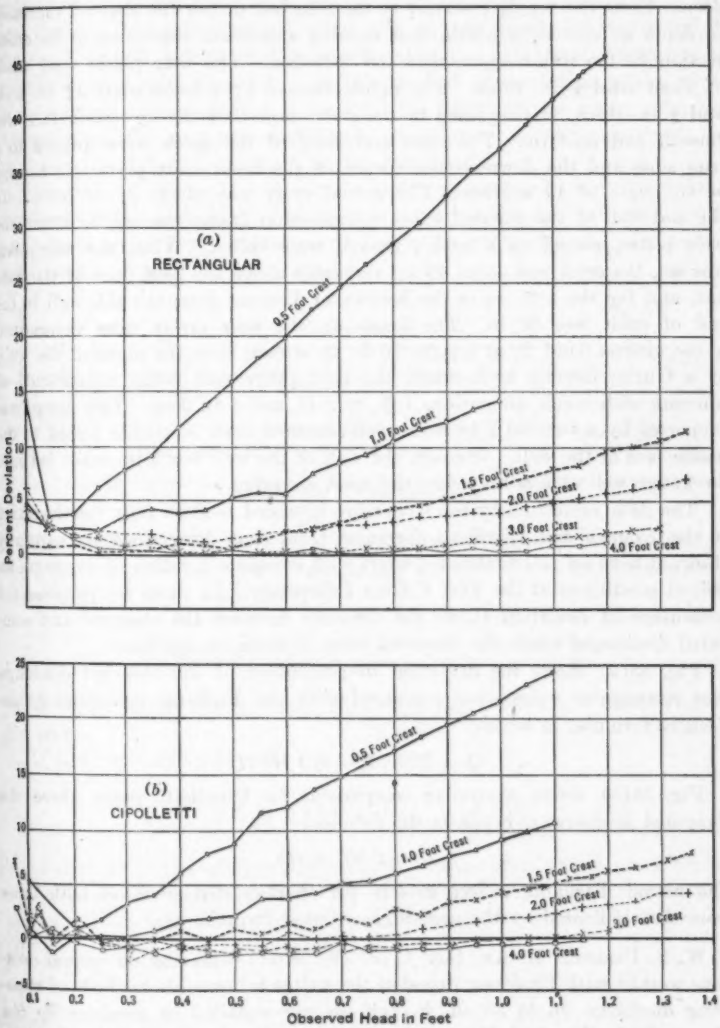

FIG. 55.—PERCENTAGE OF VARIATION OF DISCHARGE, SHARP-CRESTED WEIRS.

for depths greater than one-third the length of the crest, as recommended by Francis. The general features of this laboratory and method of conducting

the tests, together with a complete report of the results, have been described.* The weirs tested were mounted in the end wall of a concrete channel about 80 ft. long which leads to the source of the water supply. For a distance of 20 ft. back from the plane of the weir this channel is 10 ft. wide and 6 ft. deep. From the supply reservoir to the weir box proper the channel increased in depth as well as in width, thus causing a constant expansion of its cross-section as the water approached the weir box. The weir plates were made of sheet steel $\frac{1}{8}$ in. thick. The notch, formed by a brass plate $1\frac{1}{4}$ in. wide and $\frac{1}{4}$ in. thick, was so fitted to the steel sheet that the up-stream face was smooth and uniform. The crest and sides of the notch were dressed to a true adge and the down-stream corner of the brass crest plate was beveled to an angle of 45 degrees. The actual crest was about $\frac{1}{64}$ in. wide. In the end wall of the channel a heavy angle-iron frame was set, to which the weir plates, seated on a rubber gasket, were bolted. When the weir plate was set, the crest was about 42 in. vertically above the level floor of the weir box, and for the 4-ft. weirs the horizontal distance from the side-wall to the end of crest was 36 in. The heads on the weir crests were determined to the nearest 0.001 ft. at a point 10 ft. up stream from the plane of the weir, by a Gurley-Boyden hook-gauge, the hook-gauge well being constructed of concrete with inside dimensions 1 ft. by 2 ft. and 4 ft. deep. This gauge was supported by a vertical 2 by 6-in., well-seasoned stick of timber bolted to the inside face of the well. Through the wall of the weir box 1-in. tubes led into the gauge well at a point below the crest elevation.

The data submitted by the writer are intended to show only the deviation of the Francis and Cipolletti formulas from that determined by carefully observed tests on full contracted weirs with complete aeration of nappe under refined conditions at the Fort Collins Laboratory. In these comparisons the percentage of deviation shows the disparity between the observed and computed discharges where the observed value is taken as the base.

Fig. 55(*a*) shows the deviation in percentage of the observed discharge over rectangular weirs when compared with the discharge computed by the Francis formula, in which,

$$Q = 3.33\ (L - 0.2\ H)\ H^{\frac{3}{2}}$$

Fig. 55(*b*) shows a similar comparison for Cipolletti weirs where the computed discharge is based on the formula,

$$Q = 3.367\ L\ H^{\frac{3}{2}}$$

The actual discharge differs greatly for short-crested weirs of both types from that indicated by the generally accepted formula.

W. S. Pardoe,† M. Am. Soc. C. E. (by letter).—Having on several occasions worked with Professor Schoder, the writer believes his methods of measuring discharge up to 50 cu. ft. per sec. are equalled in accuracy by few laboratories and exceeded by none. His method of measuring the head, h, by means of a float and pointer reading to hundredths and interpolating

* *Journal of Agricultural Research*, U. S. Dept. of Agriculture. Vol. 5, March, 1916.

† Prof. of Hydr. Eng., Univ. of Pennsylvania,, Philadelphia, Pa.

thousandths of feet does not seem as accurate as the use of a hook-gauge with a vernier reading to thousandths and interpolating to ten thousandths.

Professor Schoder does well to call attention to the possible errors in the use of the weir in field work. An error of 2½%±, due to the condition of the crest as shown in Fig. 8 between Series I and II (Dawson), is enough to give one pause. The enormous variation due to the height of weir and to the conditions of baffling and approach make the field results obtained by means of weirs more uncertain. If this method of measuring water, which is not a cheap or easy method, is used, all the circumstances surrounding the derivation of the proposed coefficients should be carefully examined and slavishly copied if any degree of accuracy is desired.

The authors' Formula (*D*) for velocity of approach is a "jewel"; it lacks both rhyme and reason, but, as it is purely empirical, it should, like the pudding, be judged by the eating and not by its synthesis. Fig. 18 shows a remarkable accuracy for this formula.

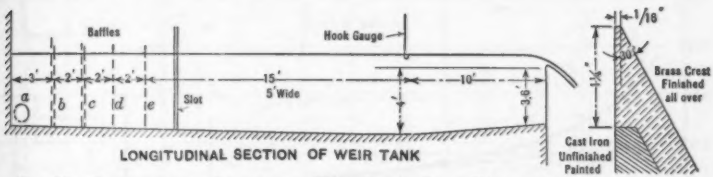

FIG. 56.—LONGITUDINAL SECTION OF WEIR TANK, CIVIL ENGINEERING DEPARTMENT, UNIVERSITY OF PENNSYLVANIA.

Francis derived his correction for velocity of approach rationally from the following expression for a rectangular orifice under a low head with an appreciable velocity of approach:

$$Q = 3.33\, L \left[\left(h_2 + \frac{v^2}{2g} \right)^{\frac{3}{2}} - \left(h_1 + \frac{v^2}{2g} \right)^{\frac{3}{2}} \right]$$

in which, h_1 and h_2 are the heads on the top and the bottom, respectively, of the orifice.

If h_1 is zero, as in the weir, this becomes:

$$Q = 3.33\, L \left[\left(h + \frac{v^2}{2g} \right)^{\frac{3}{2}} - \left(\frac{v^2}{2g} \right)^{\frac{3}{2}} \right]$$

v is the main velocity of approach, and the formula gives values much too low.

The authors cheerfully change the minus sign to plus and then re-vamp the formula to take account of the mean velocity above and below the crest, putting it in the form of Equation (*D*):

$$Q = 3.33\, L \left(h + \frac{v_a^2}{2g} \right)^{\frac{3}{2}} + L h \frac{v_b^2}{2g}$$

This formula is not homogeneous; that is, it adds peaches to pears and gets apples. The term, 3.33, by derivation includes $\sqrt{2g}$, hence the quantity involv-

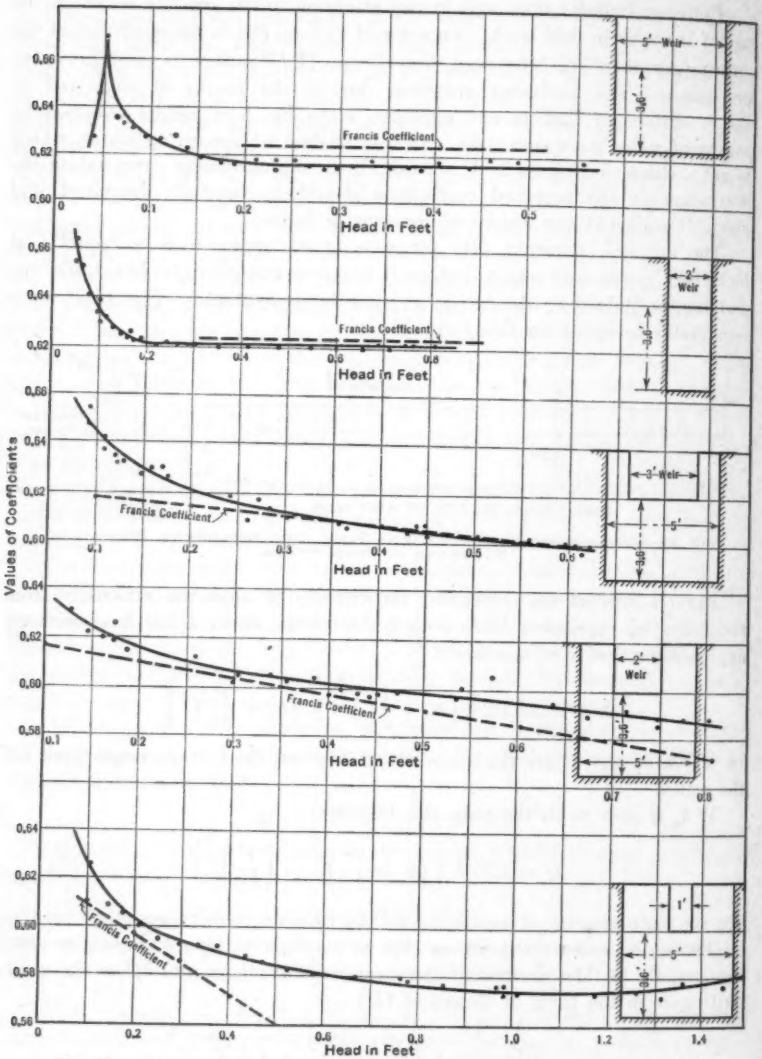

FIG. 57.—COEFFICIENTS OF RECTANGULAR WEIRS SUPPRESSED AND CONTRACTED.

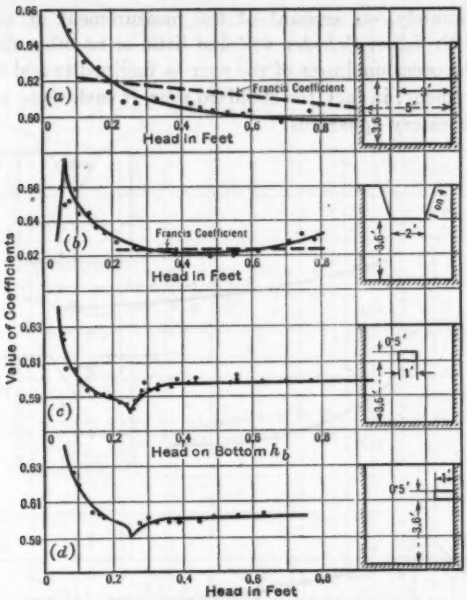

FIG. 58.—SECTIONS OF SEVERAL FORMS OF WEIRS.

ing (v_a) is $\sqrt{\frac{\text{feet}}{t^2}} \times \text{feet} \times \text{feet}^{\frac{3}{2}}$, or cubic feet per second, as it should be; but the quantity involving (v_b) is feet $\times$ feet $\times$ feet, or cubic feet. However, in spite of this, the formula gives excellent results and should be judged by

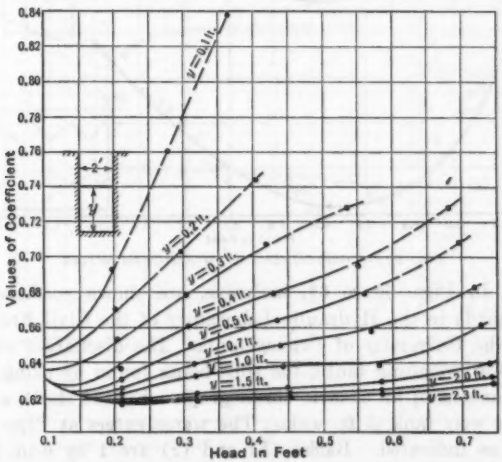

FIG. 59.

them. Unfortunately, on account of the measurement of velocities in the approach canal Professor Schoder will find little or no other data to check his own, nor will the occasional user of the weir be inclined to add to his field work by such measurement; in fact, the standard weir, if such there is, is fast becoming a bit of laboratory apparatus.

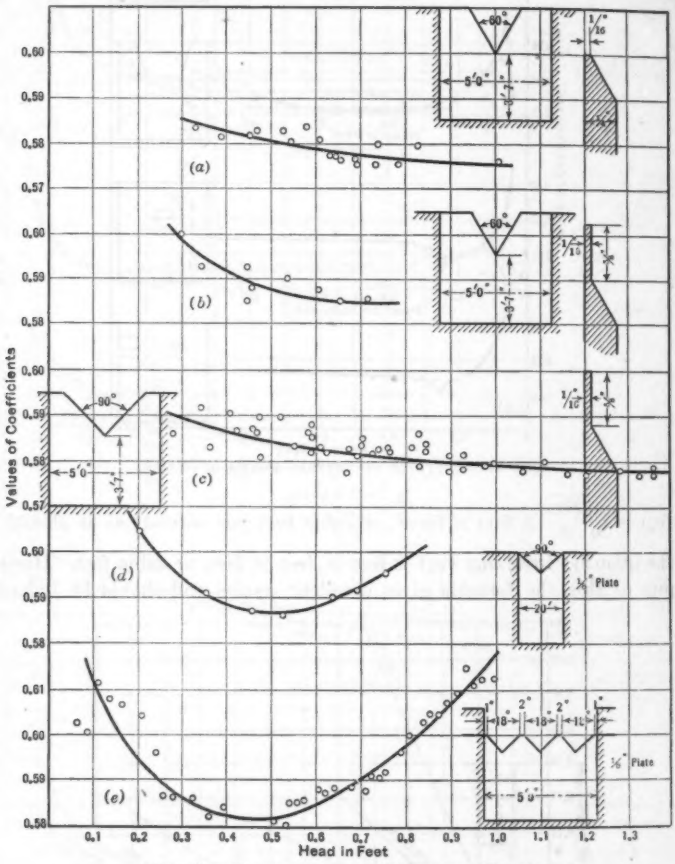

FIG. 60.—COEFFICIENTS OF V-NOTCHED WEIRS.

Herewith, in Figs. 56 to 61, inclusive, are shown coefficient curves of experiments made in the Hydraulic Laboratory of the Civil Engineering Department of the University of Pennsylvania. The discharges were measured in two 16 000-lb. weighing tanks, the time being found by using stop-watches calibrated by means of an electric chronograph. Fig. 56 shows a longitudinal section of the weir tank, 5 ft. wide. The water enters at Pipe (*a*), through drilled holes as indicated. Baffles (*b*) and (*c*) are 1 by 6-in. boards, 4 in. apart vertically and 2 in. apart horizontally. Baffles (*b*) and (*c*) are cross-

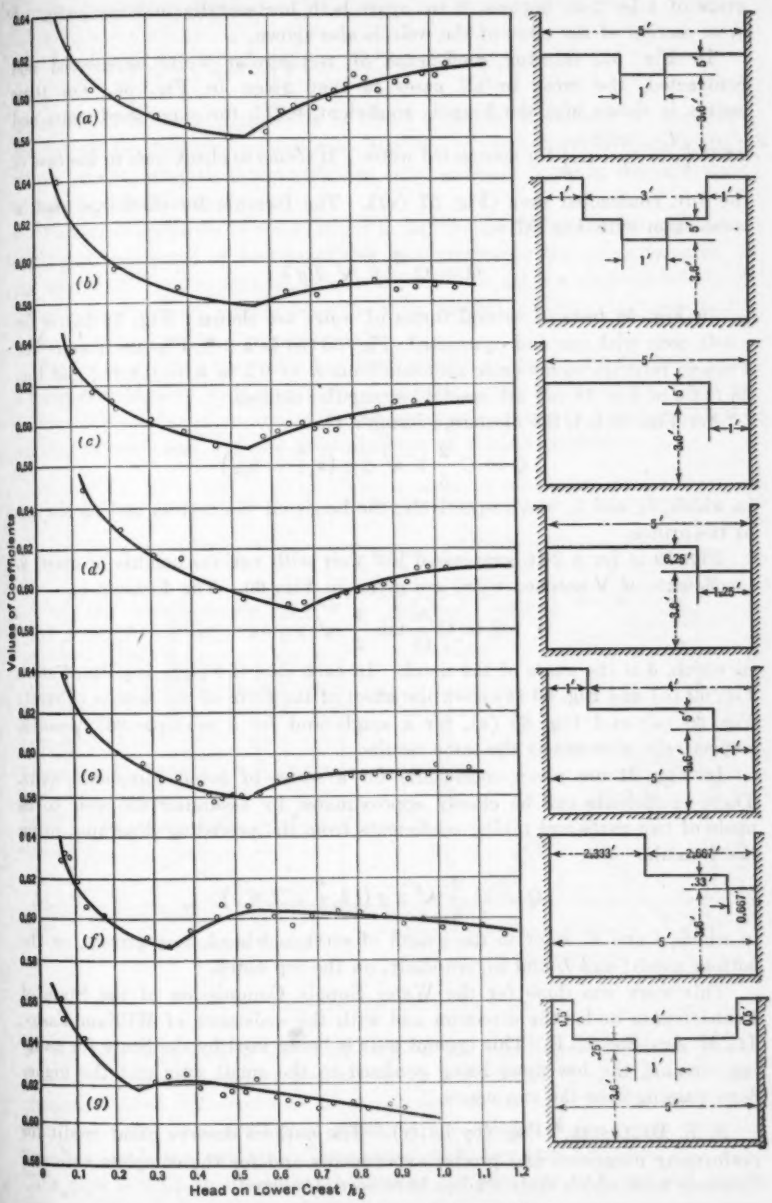

FIG. 61.—COEFFICIENTS OF COMPOUND WEIRS.

grids of 1 by 2-in. battens, 2 in. apart both horizontally and vertically. A cross-section of the crest of the weir is also shown.

In Fig. 57, showing coefficients of rectangular weirs suppressed and contracted, the crest in all cases is that given in Fig. 56. On these curves is shown also the Francis coefficient, 0.623, for suppressed weirs and $0.623\left(\frac{L-0.02\,h}{L}\right)$ for contracted weirs. It seems to check best in the case of the 3-ft. contracted weir (Fig. 57 (*e*)). The formula for discharge used in connection with Fig. 57 is,

$$Q = C\,\frac{2}{3}\,b\,\sqrt{2\,g}\,h^{\frac{3}{2}}$$

In Fig. 58 tests of several forms of weirs are shown. Fig. 58 (*a*) is for a 3-ft. weir with one end contracted; Fig. 58 (*b*) is a 2-ft. Cipolletti weir—the Francis formula seems quite accurate from $h = 0.2$ to $h = 0.8$ ft.; and Fig. 58 (*c*) and Fig. 58 (*d*) are special rectangular orifices.

For Fig. 58 (*c*), the discharge formula is,

$$Q = C\,\frac{2}{3}\,b\,\sqrt{2\,g}\left(h_b^{\frac{3}{2}} - h_t^{\frac{3}{2}}\right)$$

in which, h_b and h_t are, respectively, the heads on the bottom and on the top of the orifice.

Fig. 59 is for a 2-ft. suppressed low weir with various heights of crest, y. Coefficients of V-notched weirs are given in Fig. 60. The formula is,

$$Q = C\,\frac{8}{15}\tan\frac{\theta}{2}\,\sqrt{2\,g}\,h^{\frac{5}{2}}$$

in which, θ is the angle of the notch. In each case the plate is painted steel. Fig. 60 (*a*) and Fig. 60 (*b*) show the effect of the form of the section of crest; Fig. 60 (*d*) and Fig. 60 (*e*), for a single and for a multiple 90° V-notch, respectively, give nearly the same results.

In Fig. 61 are given coefficients for a series of seven compound weirs. These coefficients can be closely approximated by assuming the weir to be made of two parts and taking coefficients from the preceding diagrams, using the formula,

$$Q = C\,\frac{2}{3}\,\sqrt{2\,g}\left(l\,h_b^{\frac{3}{2}} + L\,h_t^{\frac{3}{2}}\right)$$

in which, l and h_b refer to the length of crest and head, respectively, on the bottom notch; and L and h_t, similarly, on the top notch.

This work was done for the Water Supply Commission of the State of Pennsylvania under the direction and with the assistance of William Easby, Jr., M. Am. Soc. C. E. This type of weir is being used by the State for gauging streams, the low flows being confined to the small weir and the higher flows passing over the two crests.

R. E. BALLESTER,* ESQ. (by letter).—The authors deserve great credit for performing numerous and precise experiments and for the complete care and clearness with which their studies have been described.

* Chf. Engr., Rio Negro Irrig. Works, Fuerte General Roca, Argentine Republic.

The tentative formula:

$$Q = 3.33\, L\left[\left(h + \frac{v_a^2}{2g}\right)^{\frac{3}{2}} + \frac{h}{3.33} \times \frac{v_b^2}{2g}\right]$$

shows that it is almost impossible to frame a general formula that takes into account all the factors that influence the flow of water over a weir, because "if the velocity conditions are unknown it is necessary to make measurements to find the distribution."

With this new series of observations it may be possible to find an expression or formula composed of two quantities, one expressing the characteristics of the weir itself, as to roughness, height, etc., and the other giving the effects of velocity conditions up stream from the weir, instead of including them in the constant, 3.33, as in the Francis formula.

With this in mind, the writer has determined the formula for discharge over a broad-crested weir with a cylindrical up-stream face (Fig. 62(*d*)), and he wishes to present the results of his observations as to the influence of the velocity of approach, and the determination of Bazin's coefficient, α.

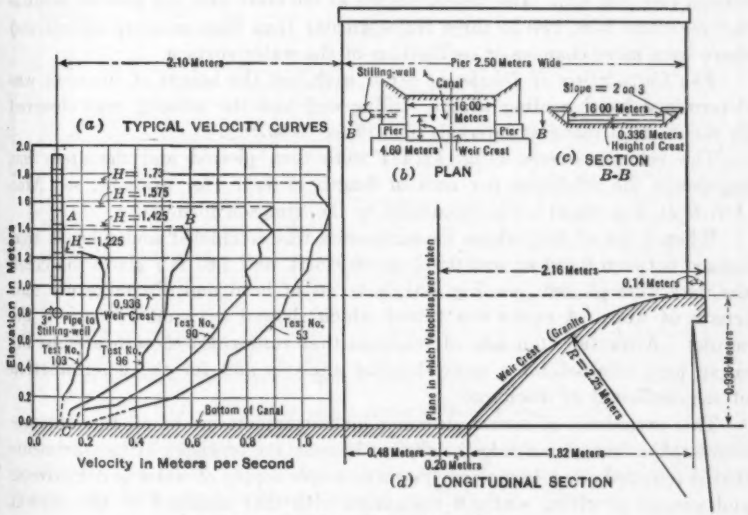

FIG. 62.—SECTIONS OF CANAL AND WEIR.

The weir is located in the main canal of the Rio Negro Irrigation Works, Argentina. Its dimensions are shown in Fig. 62(*b*), (*c*), and (*d*). The channel is lined with concrete for a distance of 200 m. (656 ft.) up stream from the weir. The scale that measures the height of the nappe over the crest is placed in a stilling-well connected with the canal by means of a pipe. This pipe is 3 in. in diameter, reduced at the entrance to 1 in., and placed 4.60 m. (15.09 ft.) up stream from the weir.

Discharge Measurements.—When the Irrigation Service was started, the discharge table was computed by means of the weir formula. The coefficient of discharge was determined from a weir with a crest similar in form to that described in the "Experiments of United States Deep Waterways Board at Cornell University, 1899".*

In attempting to control the distribution of water, some discrepancies were found, so it was decided to make discharge measurements by means of a current meter. The measurements were made by reading velocities all in the same vertical plane. At each point in the plane selected for observation, readings were taken one above the other from 0.10 m. (0.33 ft.) above the bottom of the canal (see Fig. 62).

The current meter used was an Ott Model IV. It was not possible to measure the velocity at the bottom and at the surface and these points were estimated on the curve (see Fig. 62(*a*)). The current meter was rated several times during the measurements, and no appreciable change of the constant occurred.

At first, two vertical planes for measuring velocities were used for each test, one at the vertical face at the crest and the other 2.16 m. (7.09 ft.) up stream (see Fig. 62). The measurements at the crest were not precise, because the velocities were two to three times greater than they were up stream and there were more changes or oscillations of the water surface.

The Calculation of Discharge.—For each test the height at the crest was determined by a reading at the stilling-well and the velocity was observed in the vertical plane at intervals of 0.10 m. (0.33 ft.).

The velocity curves (Fig. 62(*a*)) were then plotted and the area that represents the discharge per unit of length of weir (for example, see Area *A B C D*, Fig. 62(*a*)), was calculated by the Simpson formula.

When a set of forty-three measurements was completed under heads that ranged between 0.282 m. and 0.815 m. (0.926 ft. and 2.67 ft.) above the crest, the data were plotted (see Fig. 63) in terms of head and discharge per unit length of weir. A curve was traced which showed a visual average of the results. After this, ten sets of readings that conformed most nearly to the curve were selected for a more detailed analysis and for the determination of the coefficient of discharge.

This procedure, of course, does not make complete use of all the measurements taken, but it was judged sufficient because the precision of measurements that is required on an irrigation system of ample supply of water is not extreme and greater precision was not consistent with that obtained by the current meter measurements.

No extreme care was taken in measuring the height of water above the crest by means of floats or other devices, but methods were used throughout that were considered adequate for irrigation purposes.

The Coefficient of Discharge.—The general formula used in determining the value, *m*, was:

$$Q = m\,L\,H\,\sqrt{2\,g\,H}\left[\left(1+\frac{a\,h}{H}\right)^{\frac{3}{2}}-\left(\frac{a\,h}{H}\right)^{\frac{3}{2}}\right] \quad \ldots\ldots\ldots (1)$$

* *Water Supply Paper No. 200*, U. S. Geological Survey, Series 19, p. 90.

in which,

Q = discharge, in cubic meters per second.
m = coefficient of discharge.
L = length of weir, in meters.
H = height of water above the crest, in meters.
g = acceleration of gravity (= 9.81 m. (32.18 ft.) per sec.^{2}).
h = velocity head $\left(= \frac{U^2}{2g}\right)$.
α = coefficient to correct the influence of the different velocities in the vertical plane.
U = mean velocity in the canal at 2.16 m. (7.09 ft.) up stream from the vertical face of the crest.

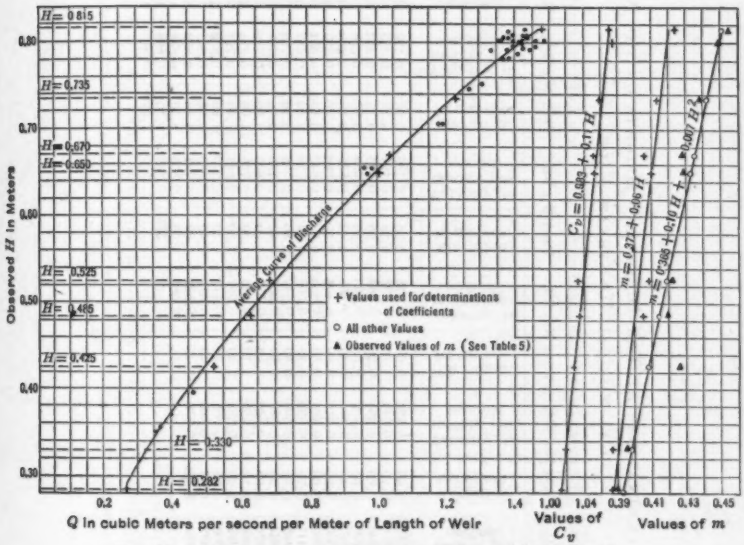

FIG. 63.—CURVES FOR THE DETERMINATION OF COEFFICIENTS.

If all water filaments had the same velocity, U, the kinetic energy would be expressed as $U^3 H$. Considering each water filament as having its own velocity, the corresponding kinetic energy is $\Sigma u^3 \Delta H$. Then,

$$\alpha = \frac{\Sigma u^3 \Delta H}{U^3 H} \quad \ldots\ldots\ldots\

TABLE 61.—VELOCITY DETERMINATIONS AT TEN DIFFERENT HEIGHTS OF THE NAPPE OVER A WEIR WITH A CYLINDRICAL UP-STREAM FACE.

Test No. 103*.		Test No. 104.		Test No. 102.		Test No. 96*.		Test No. 101.		Test No. 92.		Test No. 90*.		Test No. 88.		Test No. 52.		Test No. 53*.	
Elevation, in meters.	Velocity, in meters per second.	Elevation, in meters.	Velocity, in meters per second.	Elevation, in meters.	Velocity, in meters per second.	Elevation, in meters.	Velocity, in meters per second.	Elevation, in meters.	Velocity, in meters per second.	Elevation, in meters.	Velocity, in meters per second.	Elevation, in meters.	Velocity, in meters per second.	Elevation, in meters.	Velocity, in meters per second.	Elevation, in meters.	Velocity, in meters per second.	Elevation, in meters.	Velocity, in meters per second.
0.00	0.100	0.00	0.100	0.00	0.150	0.00	0.135	0.00	0.175	0.00	0.185	0.00	0.170	0.00	0.180	0.00	0.220	0.00	0.22
0.20	0.112	0.10	0.117	0.15	0.212	0.10	0.143	0.10	0.194	0.10	0.194	0.10	0.189	0.10	0.192	0.10	0.31	0.10	0.35
0.30	0.168	0.20	0.165	0.20	0.246	0.20	0.264	0.20	0.269	0.20	0.375	0.15	0.322	0.20	0.447	0.20	0.57	0.20	0.53
0.40	0.186	0.30	0.255	0.30	0.290	0.30	0.330	0.30	0.330	0.30	0.458	0.20	0.380	0.30	0.580	0.30	0.57	0.30	0.62
0.50	0.215	0.40	0.224	0.40	0.330	0.40	0.414	0.40	0.365	0.40	0.535	0.30	0.460	0.40	0.635	0.40	0.68	0.40	0.70
0.60	0.224	0.50	0.203	0.50	0.356	0.50	0.419	0.50	0.404	0.50	0.568	0.40	0.539	0.50	0.690	0.50	0.70	0.50	0.73
0.70	0.255	0.60	0.285	0.60	0.380	0.60	0.463	0.60	0.450	0.60	0.640	0.50	0.584	0.60	0.753	0.60	0.80	0.60	0.82
0.80	0.247	0.70	0.292	0.70	0.411	0.70	0.499	0.70	0.508	0.70	0.712	0.60	0.637	0.70	0.775	0.70	0.89	0.70	0.86
0.90	0.273	0.80	0.306	0.80	0.448	0.80	0.506	0.80	0.542	0.80	0.732	0.70	0.700	0.80	0.909	0.80	0.94	0.80	0.93
1.00	0.278	0.90	0.322	0.90	0.442	0.90	0.510	0.90	0.567	0.90	0.736	0.80	0.721	0.90	0.852	0.90	0.91	0.90	0.94
1.10	0.282	1.00	0.310	1.00	0.476	1.00	0.522	1.00	0.577	1.00	0.824	0.90	0.765	1.00	0.859	1.00	0.95	1.00	0.97
1.20	0.269	1.10	0.342	1.10	0.489	1.10	0.539	1.10	0.608	1.10	0.765	1.00	0.811	1.10	0.883	1.10	1.02	1.10	1.02
1.225	0.200	1.20	0.368	1.20	0.499	1.20	0.583	1.20	0.597	1.20	0.844	1.10	0.797	1.20	0.941	1.20	1.01	1.20	1.03
.....	.....	1.25	0.350	1.30	0.503	1.30	0 605	1.30	0.617	1.30	0.770	1.20	0.852	1.30	0.909	1.30	1.06	1.30	1.05
.....	.....	1.275	0.300	1.35	0.439	1.40	0.583	1.40	0.617	1.40	0.835	1.30	0.841	1.40	0.915	1.40	1.05	1.40	1.09
.....	.....	.....	.....	1.38	0.420	1.425	0.570	1.45	0.594	1.50	0.787	1.40	0.899	1.50	0.976	1.50	1.08	1.50	1.10
.....	.....	.....	.....	.....	.....	.....	.....	1.48	0.585	1.55	0.855	1.50	0.888	1.60	0.915	1.60	1.10	1.60	1.07
.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	1.575	0.840	1.55	0.896	1.625	0.933	1.65	1.07	1.65	1.06
.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	1.575	0.880	1.65	0.900	1.70	1.04	1.675	1.07
.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	1.73	1.04

*See Fig. 62 (a).

TABLE 62.—CALCULATION OF COEFFICIENTS, m AND C_v.

Test No. (1)	Observed head above weir, H, in meters. (2)	Distance above bottom of channel, H_1, in meters. (3)	Discharge, per meter of weir, Q, in cubic meters per second. (4)	Mean velocity of approach, $U = \frac{Q}{H_1}$, in meters per second. (5)	$U^2 H_1 = \frac{Q^2}{H_1{}^2}$. (6)	$\Sigma\,(u^2\,\Delta\,H_1)$. (7)	$\alpha = \frac{\Sigma\,(u^2\,\Delta\,H_1)}{U^2\,H_1}$. (8)	$\lambda = \frac{U^2}{2\,g}$, in meters. (9)	$\frac{\alpha\,\lambda}{H}$. (10)	$\left(\frac{\alpha\,\lambda}{H}\right)^{\frac{3}{2}}$. (11)	$1 + \frac{\alpha\,\lambda}{H}$. (12)	$\left(1 + \frac{\alpha\,\lambda}{H}\right)^{\frac{3}{2}}$. (13)	C_v = Column (13) − Column (11). (14)	Difference, Column (15) − Column (14). (16)	C_v, computed. (15)	Difference, Column (16) − Column (14). (17)	Percentage of difference. (18)	$H\sqrt{2\,g\,H}$. (19)	$m = \frac{Q}{C_v\,H\sqrt{2\,g\,H}}$ (20)	m, computed. (21)	Difference, Column (20) − Column (19). (22)	Percentage of difference. (23)	C_v = Column (14) × Column (19). (24)	m_1, computed. (25)	$\mu = \frac{3}{2} \times$ Column (24). (26)	Variation from Francis' ($\mu = 0.623$). (27)	Percentage of difference, $\frac{\text{Column (26)}}{0.623} \times 100$.
108	0.262	1.226	0.2000	0.212	0.0117	0.0146	1.248	0.0028	0.010	0.001	1.010	1.015	1.014	1.014	.....	.....	0.663	0.387	0.388	+0.001	0.3	0.392	0.393	0.589	−0.034	6.5
104	0.380	1.275	0.3317	0.36	0.0249	0.0951	1.250	0.0064	0.018	0.001	1.018	1.030	1.019	1.019	.....	.....	0.836	0.396	0.391	+0.005	0.8	0.395	0.399	0.598	−0.025	4.0
102	0.435	1.300	0.5288	0.879	0.0754	0.0915	1.212	0.0074	0.021	0.003	1.021	1.032	1.029	1.030	+0.001	0.1	1.027	0.415	0.396	−0.019	4.6	0.427	0.406	0.612	−0.011	1.8
96	0.485	1.435	0.6278	0.440	0.1818	0.1550	1.277	0.0099	0.026	0.008	1.026	1.039	1.036	1.036	+0.003	0.3	1.405	0.405	0.396	−0.009	2.2	0.419	0.415	0.622	−0.001	0.2
109	0.525	1.480	0.6966	0.467	0.1805	0.1770	1.176	0.0110	0.034	0.013	1.034	1.048	1.041	1.041	+0.006	0.6	1.685	0.409	0.402	−0.007	1.7	0.423	0.419	0.628	+0.005	0.8
92	0.650	1.675	1.0011	0.635	0.4044	0.5110	1.263	0.0205	0.040	0.024	1.040	1.061	1.053	1.054	+0.001	0.1	2.329	0.410	0.410	.....	.....	0.432	0.433	0.649	+0.026	4.2
90	0.670	1.675	1.0408	0.660	0.4540	0.5670	1.249	0.0222	0.041	0.025	1.041	1.062	1.054	1.057	+0.003	0.3	2.439	0.406	0.415	+0.009	2.2	0.426	0.435	0.652	+0.029	4.6
88	0.785	1.650	1.2347	0.743	0.6746	0.8126	1.204	0.0290	0.046	0.010	1.046	1.070	1.060	1.064	+0.004	0.4	2.791	0.414	0.415	+0.001	0.2	0.430	0.432	0.668	+0.040	6.4
52	0.800	1.700	1.4329	0.857	0.9957	1.2008	1.294	0.0377	0.055	0.018	1.055	1.084	1.071	1.071	.....	.....	3.171	0.419	0.419	.....	.....	0.449	0.449	0.673	+0.050	8.0
53	0.815	1.780	1.4740	0.852	1.071	1.2978	1.212	0.0370	0.055	0.018	1.055	1.084	1.071	1.073	+0.002	0.2	3.358	0.423	0.430	−0.008	0.7	0.453	0.451	0.676	+0.053	8.5

Table 61 shows all the velocities measured in each run. The readings in each test were taken one directly above the other and all tests were confined to the same vertical cross-section plane. The depth, H_1 (Column (3), Table 62), is somewhat different than the elevation from the bottom of the canal (that is: H + height of crest) because at the place were H_1 was measured there was a depression of the water surface. Moreover, the water surface was oscillating, so that it was difficult to take an exact measurement of the depth.

The principal values in Table 62 have been plotted in Fig. 63, as functions of H (height above the crest). The coefficient, C_v, dependent on the velocity of approach, may be represented by a linear function of H with sufficient precision, within the range of measurements. The equation of the curve is,

$$C_v = 0.983 + 0.11\ H \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

Columns (15), (16), and (17), Table 62, show that there is sufficient agreement between computed and observed values for practical purposes.

The coefficient of discharge, m, may be also represented by a linear function of H. (See Fig. 63.) This does not mean that it must be necessarily so because the number of observations is not great enough to prove it. The equation of the m-curve in Fig. 63 is:

$$m = 0.371 + 0.06\ H \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4)$$

within the range of measurements given.

The results are shown in Columns (20), (21), and (22) of Table 62. Except for $H = 0.425$, in which the percentage difference is 4.6, the variation does not exceed 1.7 per cent. It seemed a good approximation for measurements of discharge by means of a current meter, especially in view of the intended use of the weir, and, therefore, the corresponding values of C_v and m were adopted.

By multiplying these two coefficients a general coefficient, m_1, may be obtained. There is no object in doing this, however, because it obscures the separate effects of velocities of approach and the characteristics of the weir itself. This value will be (see Fig. 63):

$$m_1 = 0.365 + 0.10\ H + 0.007\ H^2 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5)$$

The variations of this formula with Francis' simple weir formula:

$$Q = \frac{2}{3}\ \mu\ L\ h\ \sqrt{2\ g\ h} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (6)$$

are given in Columns (25), (26), and (27) of Table 62, in which $\mu = 0.623$.

GARDNER S. WILLIAMS,* M. AM. SOC. C. E. (by letter).—This paper goes a long way toward settling many of the questions that have agitated experimenters dealing with the measurement of water in open channels. In the writer's judgment it is entitled to a place beside the classic experiments of Francis and Fteley and Stearns in this country and those of Bazin abroad, and in precision of measurement it exceeds them all.

To evaluate its position in the series properly it may be well to recall that quantitative knowledge of the flow of water over weirs, before the publication of this paper, was dependent on 93 experiments of Francis made at the

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Lower Locks at Lowell, Mass., in 1852; 41 experiments of Fteley and Stearns, made on the Sudbury Aqueduct, near Boston, Mass., in 1877 and 1878; and 153 experiments of Henri Bazin made at Dijon, France, in 1886. The work of Rehbock, published in 1911, however interesting, was with weirs of such small dimensions as to be hardly entitled to a place with the others, and 86 of Bazin's determinations might be logically rejected on the same ground. However, accepting all those volumetric determinations, they make a total of 287 to be compared with 1 276 made by the authors, and to these may be added the 1 162 additional volumetric measurements made by their collaborators.

The writer, as a result of his own experiments and the study of those of others, came to the conclusion long ago that the then accepted discharges of sharp-edged weirs were likely to be questioned and modified for causes at that time unknown. In preparing for publication the results of his experiments on weirs with irregular crests he expressed the coefficients for them in terms of the discharge of a sharp-edged weir under similar conditions, instead of presenting a coefficient to be introduced directly in the Francis or some other formula.* While he is not entirely sure that there may not be more in the method of reading head than the authors seem to indicate, the writer believes this paper has carried the subject of weir measurement many steps farther toward definite evaluation than anything else done in the past thirty years.

The importance of the protection of a sharp-edged weir from the effects of corrosion was apparently appreciated by Francis whose custom it was to dry the weir crest and coat it with grease at the conclusion of the day's work. Other experimenters do not seem to have been so particular. The determination of the magnitude of the corrosion effects has never been attempted before and, without the very ideal measuring apparatus used by the authors, it could not now have been successfully evaluated.

Perhaps the most interesting and valuable conclusion of the authors is that approving the use of weirs under heads greater than the height of the weir. This will often make the application of the weir as a measuring device much less difficult.

Dr. Ing. Th. Rehbock† (by letter).—Accurate and reliable measurements are among the most important requirements in the field of experimental hydraulics. In the field of practical hydraulic engineering, discharge measurements which are correct within 1 to 2% are entirely satisfactory. In field measurements in time of floods, errors of from 2 to 5% and even more are unavoidable and inconsequential. Stream flow measurements are generally made by means of current meter gaugings, although weirs are sometimes used to gauge the flow of the smaller streams.

Hydraulic Gaugings by Means of Sharp-Crested Weirs.—In experimental work the engineer is confronted not only with the problem of measuring accurately the quantity of water, but, in general, he also faces the problems of regulating the flow in such a manner that a given quantity of water is dis-

* "Hydraulic Tables," by Gardner S. Williams and Allen Hazen, Members, Am. Soc. C. E. Wiley & Sons, New York; and American Civil Engineers' Pocket Book, edited by the late Mansfield Merriman, M. Am. Soc. C. E., Wiley & Sons, New York, N. Y.

† Prof., Technische Hochschule, Karlsruhe, Germany.

charged in a unit of time. Weirs are particularly useful gauging devices in case a predetermined rate of discharge is to be maintained, for it is necessary only to fix the water level above the weir crest at an elevation which can be calculated in advance.

Weir gaugings, because of their simplicity and their accuracy over a wide range of discharge, are better adapted for use in the laboratory than any other method that has been proposed to the present time.

In the writer's opinion, the best weirs for use in field measurements are nearly triangular in cross-section with a sloping down-stream face and with rounded (preferably cylindrical) crest. This type of weir is most satisfactory because it is stable, it is not easily damaged, and its accuracy is not affected by floating material in the water. In the laboratory, however, where the factors just mentioned are of no consequence, the sharp-crested weir is to be preferred because of the greater precision of discharge measurements.

Of the sharp-crested weirs, the rectangular suppressed weir with a horizontal crest, built between vertical side walls, is particularly suitable, if the jet is fully aerated and not affected by back-water. In a few cases the triangular weir (Thomson weir) may be more satisfactory, if very small rates of discharge are to be measured. It has the disadvantage, however, that the head-discharge relation is affected to a certain extent by the shape of the crest and by the size of the approach channel in a manner that cannot be clearly determined.

The discharge over a vertical suppressed weir with a horizontal crest, as shown by experiments at Karlsruhe, Germany, is exactly proportional to the length of the crest. The rate of discharge, q, per unit length of crest is, therefore, merely a function of the two variables, h_0, the head of the weir, and p, the height of the weir crest above the bottom of the approach channel, if the following conditions are satisfied:

(*a*) The velocity in the approach channel should be as nearly the same at all points in a cross-section as practicable.
(*b*) The head should be large enough so that the jet is entirely free and does not cling to the weir plate.
(*c*) The jet should be fully ventilated so that the pressure in the space between the lower nappe and the weir plate is practically the same as that in the open air.
(*d*) The water level below the weir should be so low that the heads are not affected by back-water.

Observations made in the Karlsruhe Hydraulic Laboratory, extending over a period of more than twenty-five years, have shown no measurable fluctuations in the head on a given weir for a given constant rate of discharge, if these conditions were satisfied.

Although widely different coefficients of discharge, μ_0, have been reported by various observers for presumably similar conditions, it is the writer's opinion that discrepancies of this kind are traceable either to inaccuracies in measurements or to unsatisfactory arrangement of the testing equipment. If, for a given constant rate of discharge, fluctuating heads are observed which are not due to errors in measurements, it must be assumed either that eddies that affect the flow are formed in the approach channel, or that the

jet is not properly aerated and the nappe is depressed. The most common cause of disturbances in the flow over a sharp-crested weir, particularly under high heads, is inadequate aeration of the jet. These disturbances occur if the pipes which supply the air below the jet are too small, or if they are partly closed by water. In this case the μ_0-values are too large. It is the writer's experience that a sharp-crested, vertical, suppressed weir with adequate provisions for fully aerating the jet at all times, built between parallel side walls, is the best measuring device for hydraulic laboratories in spite of the fact that the necessity of adequate aeration adds difficulties to the construction of the weir.

The experimental layout should meet the following requirements if accurate measurements are to be assured:

1.—The approach channel should be of sufficient length and of the same width throughout.
2.—The water should enter the approach channel axially.
3.—The approach channel should be elevated high enough above the floor that the necessary measurements can be made easily and accurately, and it should be accessible on all sides so that any leaks in the channel can readily be detected and remedied.
4.—The approach channel should have a horizontal bottom and vertical, plane walls, the latter preferably made of plate glass.
5.—The up-stream face of the weir plate should be a vertical plane. It should have a horizontal crest 0.25 mm. to 1 mm. in thickness.
6.—The ventilation of the space under the jet must be adequate so that the water level above the weir is not affected by partial vacua under the jet.
7.—An auxiliary hook-gauge (with vernier scale) should be provided near the up-stream face of the weir plate to be used in transferring the height of the crest to the master gauge, that is, for determining the "zero reading" of the master gauge precisely.
8.—The master gauge with vernier scale which is used to determine the heads on the weir, should be above the draw-down curve of the water surface, or at a distance above the up-stream face of the weir at least four times the maximum head to be measured. The master gauge may be either a needle-gauge in the approach channel, or a hook-gauge in a special chamber connected with the approach channel.
9.—A sensitive float-gauge should be provided in a chamber built at one side of the approach channel to permit the reading of very small fluctuations of the water surface to fractions of 0.1 mm.

A careful and skilled observer is able to make discharge measurements with a high degree of precision if he has a measuring apparatus which fulfills, substantially at least, the requirements just enumerated, and if he uses a reliable weir formula.

The duties of the observer in making weir measurements are:

1.—To determine the length, b, of the weir crest. An accuracy of 1/2 000 is desirable.
2.—To determine the height of weir, p, above the horizontal bottom of the approach channel. An accuracy of 1/200 is sufficient.
3.—To determine the "zero reading" of the hook-gauge, or that reading for which the point of the hook is at the level of the weir crest. This determination involves the following operations:

(*a*) Bringing the point of the auxiliary hook-gauge to the elevation of the weir crest with the aid of a spirit level, and noting the gauge reading.

(*b*) Bringing the points of the auxiliary gauge and the master gauge simultaneously to a motionless water surface below the level of the weir crest and noting the readings on both gauges.

(*c*) Calculating from these measurements the "zero reading" of the master gauge, or the reading when the point of the master gauge is at the level of the weir crest.

This determination of the zero reading of the principal hook-gauge is of great importance and should be made before and after each series of tests. The precision of every measured head depends on the precision of this measurement.

4.—To determine the head on the weir by the master gauge. The point of the gauge is brought to the surface of the water in the approach channel or to the surface of the water in a chamber at one side which is connected to the approach channel by a pipe. The head, then, is the difference between the gauge reading and the zero reading.

5.—To watch the entire gauging layout very carefully at all times in order to detect any disturbances at once and to remove the causes immediately.

After the zero reading of the principal hook-gauge has been determined, it requires only a fraction of a minute to determine the head on the weir for a given quantity of flow. A skillful observer can soon learn to make these measurements quickly and accurately. Often repeated observations have shown that the heads can readily be measured within 0.1 to 0.2 mm. An exceptionally skilled observer can even, in many cases, check his measurements by repeated observations within 0.05 mm. In the case of larger quantities of flow, however, discrepancies of a few tenths of a millimeter may occur, since the water level does not remain perfectly quiet.

Formulas for Calculating the Discharge Over Sharp-Crested Suppressed Weirs Without End Contractions.—The determination of the head of water on a weir is relatively simple if one has a well-arranged layout and accurate apparatus. On the other hand, the derivation of a reliable and accurate formula for calculating quantities of discharge from these measured heads is very difficult indeed

Since the time of Poleni and Dubuat, who in the Eighteenth Century attempted to solve the problem of flow over weirs, many engineers have spent years of careful study and experiment to determine accurate formulas for the flow of water over sharp-crested weirs. The problem is a particularly difficult one, because the basic experimental work which includes volumetric measurements, must be quite precise, and because an empirical equation with three variables, h_0, p, and q, must be found, which is in substantial agreement with the results of the test.

All well known formulas which have been used in the past are of the type proposed by Poleni and Dubuat, namely:

$$Q = \frac{2}{3} \mu_0 \sqrt{2g}\, b\, h_0^{\frac{3}{2}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (7)$$

This formula is based on the fallacious assumption that water flows through the entire cross-section, $F = b\ h_0$, over the weir crest, whereas the actual cross-section of the jet, because of the draw-down which is equal to approximately 0.15 h_0, is only about $F' = 0.85\ b\ h_0$. In the derivation of this formula a pressure distribution within the jet itself is assumed, which is altogether different from that which exists in the actual jet. The fact that, in the case of a sharp-crested weir, the nappe rises to a point about 0.11 h_0 above the weir crest is neglected entirely. Despite the illogical assumptions on which Formula (7) is based, it is adaptable because all errors in the derivation can be nullified by the proper selection of the discharge coefficient, μ_0. As a consequence it has remained the foundation of all well known weir formulas.

Weisbach attempted to improve the formula by expressing μ_0 as a function of the depth of flow in the approach channel, or, what amounts to the same thing, as a function of the mean velocity of approach. His formula (1844) is of the type:

$$\mu_0 = \mu \left\{ a + b \left(\frac{h_0}{p + h_0} \right)^2 \right\} \quad \ldots\ldots\ldots\ldots\ldots\ldots (8)$$

in which, μ is the coefficient for a very high weir, and a and b are constants. Most of the weir formulas which have subsequently been proposed are of the Poleni-Dubuat-Weisbach form and differ essentially only in the values of μ, a and b.

The following formulas for μ_0 in terms of h_0 and p, in meters, are those most commonly used in Europe:

Bazin's formula (1888):

$$\mu_0 = \left(0.6075 + \frac{0.0045}{h_0} \right) \left\{ 1 + 0.55 \left(\frac{h_0}{h_0 + p} \right)^2 \right\} \quad \ldots\ldots\ldots (9)$$

Frese's formula (1890):

$$\mu_0 = \left(0.615 + \frac{0.0021}{h_0} \right) \left\{ 1 + 0.55 \left(\frac{h_0}{h_0 + p} \right)^2 \right\} \quad \ldots\ldots\ldots (10)$$

The formula of the Swiss Society of Engineers and Architects (1924):

$$\mu_0 = 0.615 \left(1 + \frac{1}{1\,000\ h_0 + 1.6} \right) \left\{ 1 + 0.5 \left(\frac{h_0}{h_0 + p} \right)^2 \right\} \quad \ldots\ldots (11)$$

The writer's formula (1912):

$$\mu_0 = 0.605 + \frac{1}{1\,050\ h_0 - 3} + 0.08 \frac{h_0}{p} \quad \ldots\ldots\ldots\ldots\ldots (12)$$

The writer had tried in vain for many years to determine a reliable discharge formula of the Weisbach type which agreed with the results of experiments made in the Karlsruhe Hydraulic Laboratory, even on weirs with small values of p and h_0. Formula (12), which was finally proposed, differs from the type presented by Weisbach, but is in close agreement with the writer's tests.

For heads greater than 0.03 m., that is, for practically all cases which are met in practice, this formula can be simplified into:

$$\mu_0 = 0.605 + \frac{1}{1\,000\ h_0} + 0.08 \frac{h_0}{p} \quad \ldots\ldots\ldots\ldots\ldots (13)$$

The differences between the μ_0-values of Formulas (12) and (13) are approximately 0.25% for $h_0 = 0.03$ m., while for $h = 0.04$ m., this discrepancy between the formulas is only about 0.10% and becomes still smaller for higher heads.

Strictly speaking, Formulas (9) to (13) are not dimensionally correct. Some disturbance, which is not yet clearly understood, arises in the case of flow of water over sharp-crested weirs, which adds materially to the difficulties of determining a formula for the coefficient of discharge. In consequence of this disturbance the law of similarity cannot be strictly applied in the case of sharp-crested weirs because the coefficient, μ_0, is not exactly the same for any two sharp-crested weirs of different absolute heights, p, even if the ratios, $\frac{h_0}{p}$, are identical.

The fact that the writer's discharge formula,

$$Q = \frac{2}{3}\left(0.605 + \frac{1}{1\,050\,h_0 - 3} + 0.08\,\frac{h_0}{p}\right)\sqrt{2\,g}\,b\,h_0^{\frac{3}{2}} \quad \ldots\ldots(14)$$

determined from the results of experiments, gives for the metrical system values almost identical with those obtained from the formula,

$$Q = \frac{2}{3}\left(0.602 + 0.0832\,\frac{h}{p}\right)\sqrt{2\,g}\,b\,(h_0 + 0.00125)^{\frac{3}{2}} \quad \ldots\ldots(15)$$

may lead to a possible explanation. A comparison of these two formulas indicates that the disturbance influences the discharge in such a way that the discharge is not directly proportional to the $\frac{3}{2}$ power of the head measured on the master gauge, but rather to that of an effective head which is about 1.25 mm. greater than that actually observed.

L. Prandtl has expressed the opinion that capillarity might be responsible for the form of the second term of Formula (12) and that the behavior may possibly be comparable to the rise of the water surface along a vertical wall, $h' = \sqrt{\frac{K}{\gamma}}$, in which, K is the coefficient of capillarity and γ, the density of the fluid.

Since h' is equal to 0.00267 m., Formula (12) may be changed accordingly to read:

$$\mu_0 = 0.605 + \frac{0.36}{h_0\sqrt{\frac{\gamma}{K}} - 1} + 0.08\,\frac{h_0}{p} \quad \ldots\ldots(16)$$

The values obtained from Formula (16) are only about $\frac{1}{3}$ of 1% less than those obtained from Formula (12) for heads of 0.01 m., and the difference between the formula values does not exceed 0.25% for heads greater than 0.015 m. These discrepancies are meaningless in the case of such low heads and practically disappear at higher heads.

Formula (15) may also be rewritten to take account of the capillarity coefficient, as follows:

$$Q = \frac{2}{3}\left(0.602 + 0.0832\,\frac{h_0}{p}\right)\sqrt{2\,g}\;b\left(h_0 + 0.47\sqrt{\frac{K}{\gamma}}\right)^{\frac{3}{2}} \ldots\ldots (17)$$

It is evident that Formulas (15), (16), and (17) are dimensionally correct, because $\sqrt{\frac{K}{\gamma}}$ is a length.

It is not yet definitely known, however, whether this disturbance is due to capillarity or to some other cause. In any event some phenomena exist which have the same effect on the discharge as a constant increase of head on the weir equal to 1¼ mm.

Formula (15) can be simplified and expressed in second-feet (with sufficient exactness) as follows:

$$Q = b\left(3.22 + 0.445\,\frac{h_0}{p}\right)(h_0 + 0.004)^{\frac{3}{2}}\ \text{sec-ft} \ldots\ldots\ldots\ldots (18)$$

Formulas (9), (10), and (11) have been considered applicable with the following limitations:

(*a*) Formula (9):

$$0.1 < h_0 < 0.6\ \text{m.}$$

(*b*) Formula (10):

$$0.1 < h_0 < 0.6\ \text{m.}$$

(*c*) Formula (11):

$$0.025 < h_0 < 0.8\ \text{m.};\ p \geqq 0.3\ \text{m.};\ h_0 < p$$

Formulas (12) to (19), inclusive, are not necessarily restricted between any given limits. They apply in general to any sharp-crested vertical weir without end contractions as long as the jet is perfectly free, fully aerated, and not affected by back-water. Since a free overfall does not exist for heads less than about 0.01 m., this head is automatically the lower limit of applicability of the formulas. Obviously, the accuracy of measurements with heads so small is not very great.

The formulas are applicable even to weirs of very low height, p, and have been found to agree with the tests of a weir only 0.056 m. high.

These formulas seem applicable, too, for all great heights and large heads in practical use, as long as it is possible to aerate the nappe fully. This may be deduced from the law of similarity. However, the writer advises against building sharp-crested weirs higher than 1.2 m. (4 ft.), for gauging purposes, and using such weirs for heads much greater than 0.6 m. (2 ft.).

If the bottom of the channel below the weir is at the same level as the bottom of the approach channel, the heads, h_0, on the weir should not be much greater than the height of the weir, p, because there is danger of the heads being affected by the back-water in the discharge channel. If the bottom of the discharge is lower than that of the approach, the formulas are in substan-

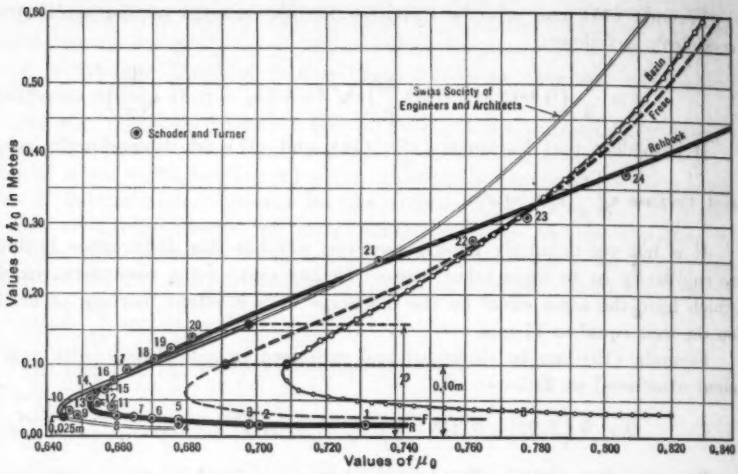


FIG. 64.—μ_0-VALUES FOR WEIRS 0.5 FEET HIGH.

tial agreement with the tests even if the heads are two to four times the weir height and perhaps still more. This may be seen clearly in Fig. 64.

The best agreement between the formulas and tests may be expected, however, if 0.1 m. $< p <$ 1.00 m. and 0.025 m. $< h_0 <$ 0.60 m. Weirs, therefore, should preferably be designed so that the heights and heads, under which they

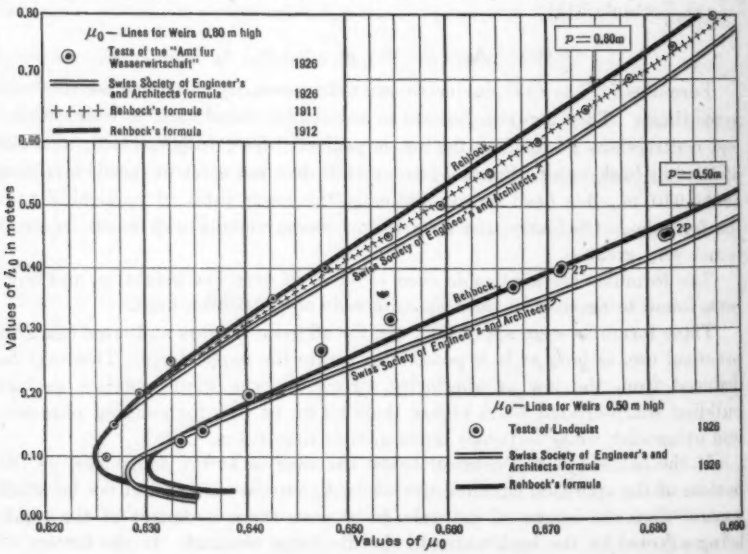


FIG. 65.—COMPARISON OF THE μ_0-VALUES.

will be used, fall between these limits. In general, this can be attained through proper selection of the length of the weir crest. Weirs of ordinary spillway section with a cylindrical crest are to be preferred to sharp-crested weirs if very large rates of discharge are to be measured, for if the discharge over a sharp-crested weir becomes very great, it is difficult to provide adequate ventilation of the jet.

The tests of the Swiss Government Department of Hydraulics have shown that weir measurements for heads of 0.8 m. are still reliable and Fig. 65 shows that the writer's formulas give values differing from these measurements by only fractional parts of 1% up to heads of 0.8 m.

Comparison of Formulas in Use in Europe with Recent Tests.—The writer has restricted himself to a comparison of the four formulas for the flow of water over sharp-crested weirs, which are in common use in Europe (Formulas (9), (10), (11) and (12)). A comparison of the formulas of Francis, Fteley and Stearns, and H. W. King, M. Am. Soc. C. E., which are commonly used in the United States, has not been made.

Relatively few new tests have come to the writer's attention in recent years. Most of them were for only one height of weir and, therefore, did not permit a thorough comparison. Since the publication of the writer's formulas, the only European tests which have come to his knowledge with enough information, so that the μ_0-values could be calculated from the data are those of Fr. Schaffernak, in Vienna, Austria (1915), Erik G. W. Lindquist, M. Am. Soc. C. E., in Stockholm, Sweden (1926), and the Swiss Government Department of Hydraulics (1926). All these gaugings were made with the greatest care, but with different equipment and different rating methods.

Schaffernak published* the results of seventeen measurements for a weir 1 m. long and 0.56 m. high for mean heads lying between 0.0288 m. and 0.3082 m. In these tests only one ventilator pipe with a cross-sectional area of about 5 sq. cm. was provided for aerating the space under the jet. This pipe certainly could not have provided sufficient air for heads greater than 0.2 m. Therefore, only the data for heads less than 0.2 m. may be considered to be correct. The results of these eight tests are shown in Table 63.

The maximum difference between the μ_0-values calculated from Formula (12) and those observed by Schaffernak for heads between 0.0288 m. and 0.187 m., is only 0.0027, or 0.43 per cent. The mean absolute percentage differences of observed and computed values, according to Formula (12), is 0.16.

Lindquist's experiments,† in Stockholm, were made on a weir, 0.50 m. in height, for heads that ranged between 0.124 m. and 0.452 m. (Table 64).

The maximum difference between observed μ_0-values and those of the writer's formula is 0.0042, or 0.64 per cent. The mean of all percentage differences is 0.28.

* Mitteilungen der Versuchsanstalt für Wasserbau über ausgeführte Versuche, Wien, 1915, 1. Folge B, "Die Eichung eines scharfkantigen belüfteten Ueberfallwehres ohne seitliche Strahleinzwängung (Rehbock'scher Ueberfall)".

† Sent to the writer by letter of April 3, 1926.

The measurements of the Swiss Department of Hydraulics, in Berne,* agree equally well with the results of Formula (12). These measurements were made on a sharp-crested weir, 0.8 m. high, at the Amsteg Power Plant, in Switzerland, and the data were given to the Swiss Society of Engineers and Architects to be used in preparing standard specifications for weir measurements.

TABLE 63.—SCHAFFERNAK'S EXPERIMENTS ON A WEIR 0.56 METER HIGH.

No.	h_0, in meters.	μ_0', according to tests of Schaffernak.	μ_0, according to Formula (12) of Rehbock.	Difference, $\Delta\mu_0 = \mu_0' - \mu_0$.	Difference, $\Delta\mu_0$, percentage.
1	0.0288	0.6444	0.6458	−0.0014	−0.22
2	0.0488	0.6313	0.6327	−0.0014	−0.22
3	0.0878	0.6301	0.6308	−0.0007	−0.11
4	0.1427	0.6295	0.6322	−0.0027	−0.43
5	0.1583	0.6330	0.6337	−0.0007	−0.11
6	0.1693	0.6354	0.6349	+0.0007	+0.11
7	0.1777	0.6360	0.6358	+0.0002	+0.03
8	0.1890	0.6374	0.6371	+0.0003	+0.05
Arithmetic mean	.....	.....	.....	− 0.0007	−0.11
Mean discrepancy	.....	.....	.....	0.0010	0.16

Table 65 shows the mean values of μ_0, which are presented in Table 26 of the original report of the Swiss Department of Hydraulics previously mentioned, and the corresponding μ_0-values according to Formula (12).

TABLE 64.—LINDQUIST'S EXPERIMENTS ON A WEIR 0.50 METER HIGH.

No.	h_0, in meters.	μ_0', according to tests of Lindquist.	μ_0, according to Formula (12) of Rehbock.	Difference, $\Delta\mu_0 = \mu_0' - \mu_0$.	Difference, $\Delta\mu_0$, percentage.
1	0.1242	0.6335	0.6327	+0.0008	+0.13
2	0.1400	0.6357	0.6343	+0.0014	+0.22
3	0.1972	0.6403	0.6415	−0.0012	−0.19
4	0.2685	0.6475	0.6516	−0.0041	−0.63
5	0.3165	0.6545	0.6587	−0.0042	−0.64
6	0.3686	0.6668	0.6666	+0.0002	+0.03
7	0.3977	0.6714	0.6710	+0.0004	+0.06
8	0.3983	0.6715	0.6711	+0.0004	+0.06
9	0.4508	0.6820	0.6793	+0.0027	+0.40
10	0.4517	0.6825	0.6794	+0.0031	+0.46
Arithmetic mean	.....	.....	.....	−0.00005	−0.01
Mean discrepancy	.....	.....	.....	0.00185	0.28

The mean discrepancy between observed and computed values of μ_0 is 0.25 per cent. The coefficients of discharge reported by the Swiss Department of Hydraulics agree still more closely with the values of μ_0 computed by the writer's earlier formula (1911)†:

$$\mu_0 = 0.605 + \frac{1}{1\,100\,h_0} + \frac{h_0}{12\,p} \qquad (19)$$

* Mitteilungen des Eidgenössischen Amtes für Wasserwirtschaft, herausgegeben unter Leitung von Dr. Mutzner, 1926, No. 18.

† *Zeitschrift* des Verbandes deutscher Architekten- und Ingenieur Vereine, 1, Jahrgang No. 1, 6 Januar, 1912.

The greatest discrepancy between the observations and the results of Formula (19) do not in any case exceed 0.0024, and the mean discrepancy between all observed and computed values is only 0.0009, or 0.14 per cent. A comparison of the μ_0-values of these observations and those of Lindquist with the writer's μ_0-lines for the same heights of weir, is shown in Fig. 65.

A comparison of the coefficients of discharge determined experimentally in Vienna, Stockholm, and Amsteg for different testing layouts, with the corresponding coefficients calculated by Formula (12), shows a maximum discrepancy of 0.64 per cent. The mean discrepancy for the entire thirty-three measurements is only 0.24 per cent.

TABLE 65.—EXPERIMENTS AT AMSTEG BY THE SWISS DEPARTMENT OF HYDRAULICS.

No.	h_0, in meters.	μ_0' according to tests of Swiss Department of Hydraulics.	μ_0, according to Formula (19) of Rehbock.	Difference, $\mu_0 = \mu_0' - \mu_0$.	Difference, μ_0, in percentage.	μ_0, according to Formula (17) of Rehbock.	Difference, $\mu_0 = \mu_0' - \mu_0$.	Difference, μ_0, in percentage.
1	0.1	0.6262	0.6248	+0.0014	+0.22	0.6252	+0.0010	+0.16
2	0.15	0.6269	0.6265	+0.0004	+0.06	0.6271	−0.0002	−0.03
3	0.2	0.6294	0.6298	−0.0004	−0.07	0.6307	−0.0013	−0.21
4	0.25	0.6323	0.6338	−0.0015	−0.24	0.6347	−0.0024	−0.38
5	0.3	0.6376	0.6382	−0.0006	−0.10	0.6295	−0.0019	−0.30
6	0.35	0.6428	0.6427	+0.0001	+0.01	0.6442	−0.0014	−0.22
7	0.4	0.6489	0.6474	+0.0015	+0.23	0.6491	−0.0002	−0.03
8	0.45	0.6542	0.6521	+0.0021	+0.32	0.6541	+0.0001	+0.02
9	0.5	0.6596	0.6569	+0.0027	+0.40	0.6590	+0.0006	+0.09
10	0.55	0.6646	0.6617	+0.0029	+0.43	0.6640	+0.0006	+0.09
11	0.6	0.6694	0.6666	+0.0028	+0.41	0.6691	+0.0003	+0.05
12	0.65	0.6741	0.6715	+0.0026	+0.39	0.6742	−0.0001	−0.02
13	0.7	0.6787	0.6764	+0.0023	+0.34	0.6793	−0.0006	−0.09
14	0.75	0.6832	0.6813	+0.0019	+0.29	0.6844	−0.0012	−0.18
15	0.8	0.6877	0.6862	+0.0015	+0.22	0.6895	−0.0018	−0.26
Arithmetic mean	....	.....	.....	+0.0013	+0.20	.....	−0.0006	−0.09
Mean discrepancy....	....	.....	.....	0.0016	0.25	.....	0.0009	0.14

Referring to the form of the μ_0-lines in Fig. 65, it is noteworthy that the writer, in setting up Formulas (12) and (19), took into consideration only observations for weirs 0.5 m. or less in height and for heads up to 0.18 m. in magnitude for the latter and up to 0.231 m. for the former.

Even if the μ_0-values for larger weir and overflow heights were obtained only by consideration of the law of similitude as a basis, they show a variation of less than 0.25% from the μ_0-values later obtained by the Swiss Department of Hydraulics on weir and overflow heights of 0.80 m.

Experiments were also conducted in the Hydraulic Laboratory at Dresden for the purpose of checking Formula (12), but the details of these tests were not published. A letter from H. Engels, Director of this Laboratory, stated, however, that the agreement of the formula with the experimental results was most satisfactory.

M. Carstanjen* shows discharge curves for two series of experiments made by the late A. Koch in Darmstadt and the corresponding discharge curves according to Formula (12). The experimental and calculated curves lie near together, but the data are not presented in sufficient detail, so that a numerical comparison of the μ_0-values can be made.

In addition to these more recent experimental verifications in the various hydraulic laboratories of Europe, the writer has also a report from Dr. Hahn, of the J. M. Voith Company, in Heidenheim, Germany, that Formula (12) agreed very well with experimental results obtained in the Voith Turbine Laboratory.

The extensive gaugings of the flow over sharp-crested weirs made in recent years by the Swiss Society of Engineers and Architects have not yet been published and, therefore, could not be included in the comparisons which are presented herewith.

The experiments† by Floyd A. Nagler, M. Am. Soc. C. E., for a weir 1.124 m. high, gives μ_0-values still higher than those of Bazin and from 3.6 to 4.9% higher than those of Formula (12). These results show that the hydro-chemical method is not reliable for verifying weir measurements.

Weir Gaugings by Schoder and Turner.—The authors' paper, presenting the results of 2 438 gaugings, is the most comprehensive compilation of the records of gauging of sharp-crested suppressed weirs that has come to the writer's attention. It includes the results of experiments on weirs of different heights, under wide ranges of heads, and is a source of information of inestimable value to the hydraulic experimenter.

After a detailed study of the 1 017 measurements in Series D to P (Table 1), the writer has come to the conclusion that these measurements for weirs of normal height (0.5 ft. to 4 ft.) under normal heads between 1 in. and 2 ft. were made with a particularly high degree of accuracy. For heads less than 1 in. (0.025 m.), the data are somewhat erratic, but this is considered unavoidable because measurements of heads could only be made by means of the float-gauge without a vernier scale and, therefore, could not reach a very great precision.

The accuracy of the data for the weirs, 5.5 and 7.5 ft. in height, is open to some question. If the coefficients of discharge, μ_0, are plotted as abscissas and the corresponding heads, h_0, as ordinates for the weirs of different heights, p, it will be seen that the coefficient of discharge for a given head decreases regularly as a function of the height for weirs measuring 4 ft. or less. However, some of the coefficients of discharge for weirs 5.5 and 7.5 ft. high are greater than those for the weir 4 ft. high under the same heads. It is wholly unreasonable to think that, for a given head, h_0, the coefficient of discharge becomes greater when the weir height is increased because the velocity of approach must become increasingly slower.

The erratic behavior of the data for the weirs, 5.5 and 7.5 ft. in height, is at once traceable to the construction of the approach channel shown in Fig. 6.

* Koch-Carstanjen, "Von der Bewegung des Wassers und den dabei auftretenden Kräften", Berlin, 1926, Julius Springer Verlag, p. 156.

† "Verification of the Bazin Weir Formula by Hydro-Chemical Gaugings," *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919-20), p. 105.

The surface of the fixed bottom of the approach channel over the 30-in. supply pipe appears to be only about 5 ft. below the weir crest, while the movable bottom near the weir lies below this level during the tests of the two highest weirs. Undoubtedly, the disturbance arising from this type of construction changed the flow in such a way that the "effective" height of the weir was actually much less than the height measured from the crest to the bottom of the approach channel near the weir. The fact that the data for the 5.5 and 7.5-ft. weirs (Series D, K, L, M, and O, Table 1) are obviously unreliable is, however, of little importance, because sharp-crested weirs more than 4 ft. in height are almost never used in practice.

The writer also feels that the accuracy of the data for heads greater than 0.5 to 0.6 m. (1⅔ to 2 ft., and more) is also questionable. There are disturbances in the law of similarity at low heads, but these are barely perceptible if the head on the weir is about 0.3 m. (1 ft.), and they disappear almost entirely at higher heads. If the μ_0-values of Schoder and Turner for high heads were correct some new disturbance must occur which makes the coefficients of discharge increase suddenly if the head is greater than about 2 ft. This is hardly to be expected and is not borne out by the experiments of the Swiss Government Department of Hydraulics which have been summarized in Table 65 and Fig. 65.

The most common cause of such abrupt changes in the discharge coefficient-head relation at the higher heads is insufficient ventilation of the space between the jet and the weir plate. Whether the sudden divergence of the authors' measurements from the $\mu_0 h$-curve, which is nearly straight, is chargeable to insufficient ventilation, or to some other cause, cannot be stated definitely without having at hand more detailed information concerning the arrangement of the testing layout.

If the measurements for heads greater than 0.6 m. (2 ft.) are neglected and the formulas are applied only to heads less than 2 ft., as Bazin and Frese have recommended, the comparison still covers nearly the entire practical range of heads, which is generally used for sharp-crested weirs.

The coefficient of discharge, μ_0, in Formula (7) was calculated for each of the Schoder and Turner gaugings for all heads between 0.01 m. and 0.60 m. These μ_0-values are compared with the corresponding μ_0-values determined by Formulas (9) to (12), inclusive. Figs. 66 to 71 show graphically the comparison of the observations for weirs ¾, 1, 1½, 2, 3, and 4 ft. high, respectively, as found by Formulas (9), (10), (11), and (12). (See, also, Fig. 64.) The limits of the formulas, as given by their respective authors, are shown by black points. Beyond these limits the curve of the formula is shown only by lighter lines.

These diagrams show that, in all cases, Formula (9) gives much higher values of discharge for a given head than the Schoder and Turner tests indicate. Formula (10) shows a somewhat better agreement, but all the values are still too high. Formula (12) shows the best agreement with the results of the authors' experiments. Formula (11) also agrees quite well with the

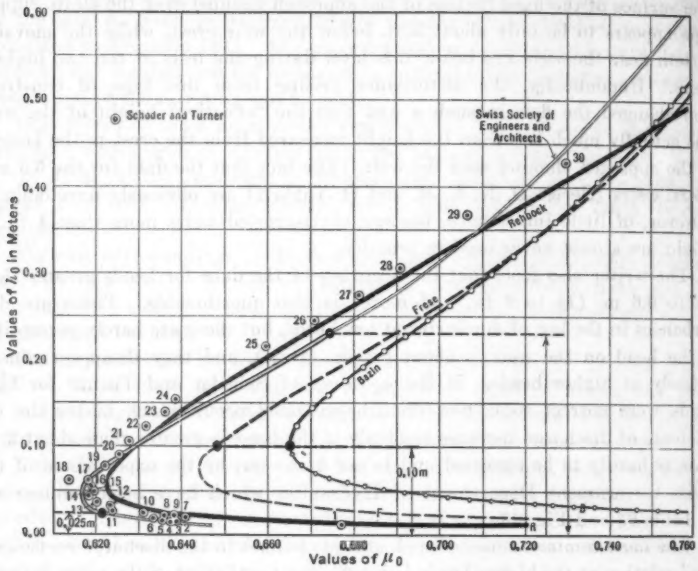


FIG. 66.—μ_0-VALUES FOR WEIRS 0.75 FOOT HIGH.

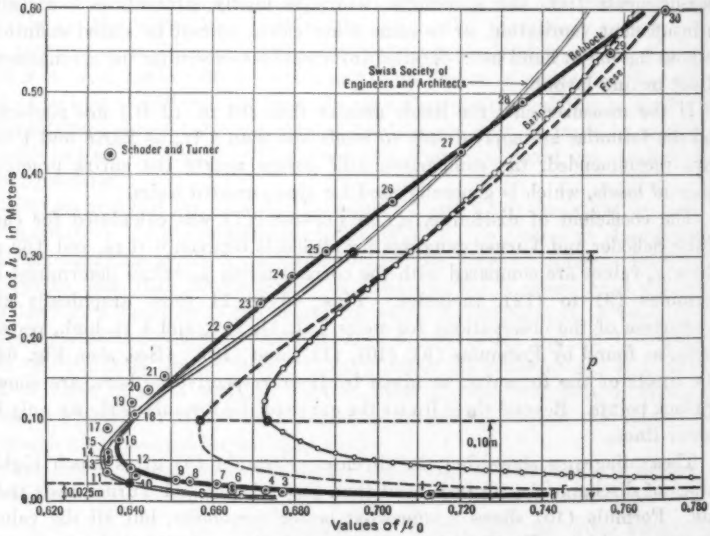


FIG. 67.—μ_0-VALUES FOR WEIRS 1.0 FOOT HIGH.

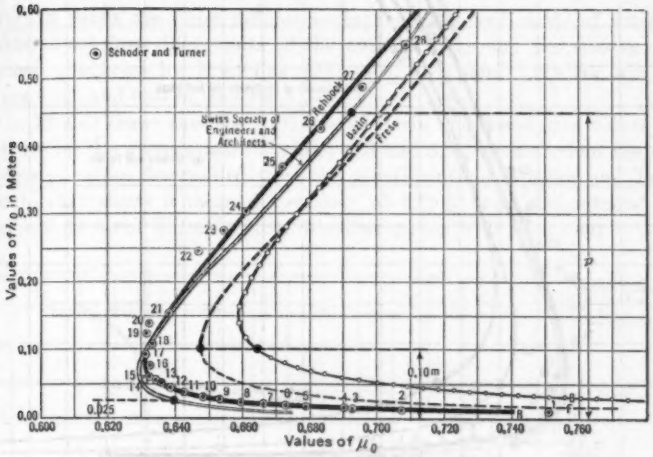


FIG 68.—μ_0-VALUES FOR WEIRS 1.5 FEET HIGH.

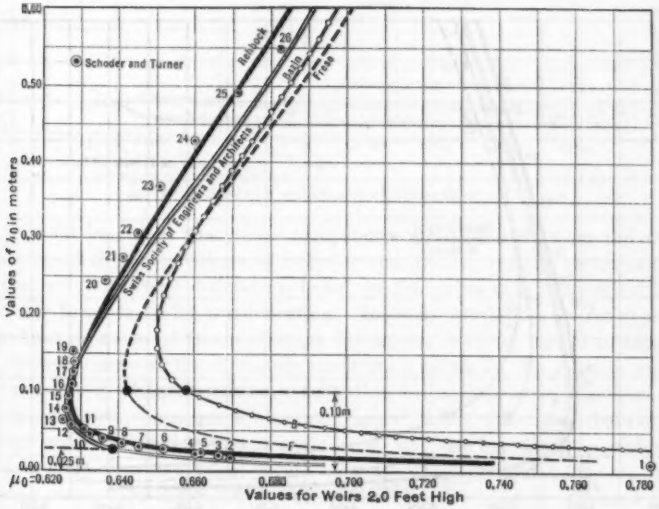


FIG. 69.—μ_0-VALUES FOR WEIRS 2.0 FEET HIGH.

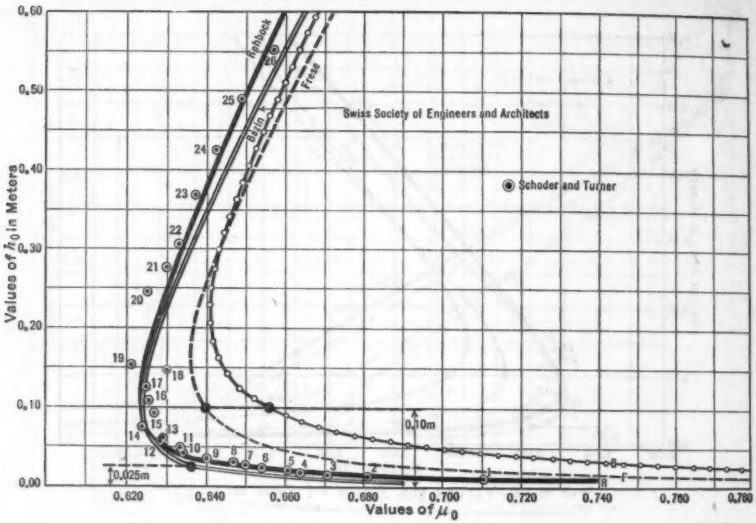


FIG. 70.—μ_0-VALUES FOR WEIRS 3.0 FEET HIGH.

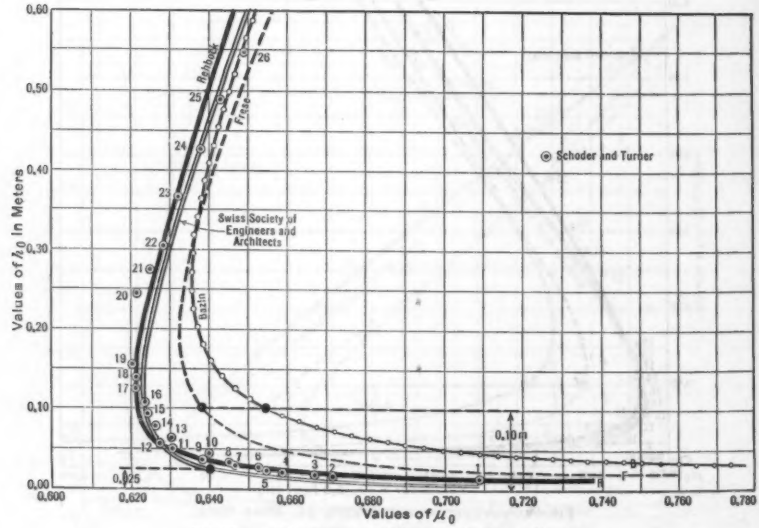


FIG. 71.—μ_0-VALUES FOR WEIRS 4.0 FEET HIGH.

experimental results, but shows mean differences nearly twice as large as those from the writer's Formula (12).

Fig. 72 shows the mean differences between the coefficients of discharge, μ_0, determined from the results of the authors' tests and the corresponding μ_0-values calculated by Formulas (9), (10), (11) and (12) for all heads between 0.025 and 0.60 m. and for $h_0 < 2\,p$.

Fig. 72 also shows that the formulas of Bazin and Frese give values of μ_0 which are on the average approximately 6.3 and 3.2% greater than the corresponding μ_0-values determined from the experiments of Schoder and Turner. Formula (11) shows a mean discrepancy of 0.70% from the authors' tests, while, in the case of Formula (12), this discrepancy is 0.38 per cent.

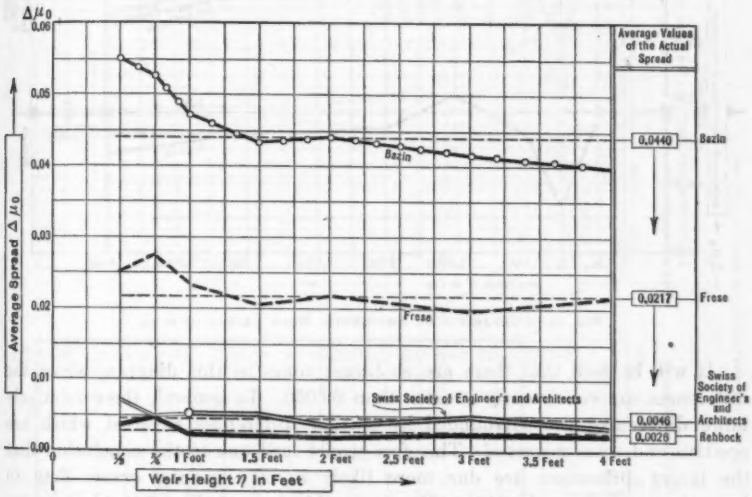


FIG. 72.—COMPARISON OF AVERAGE VALUES OF Δ μ_0.

It should be stated that the discrepancies mentioned between the several formulas and the observations of Schoder and Turner are the mean values of the absolute percentage differences based on the value of μ_0 according to the particular formula under consideration. In other words the figures represent the arithmetic mean of the percentage deviations, without regard to algebraic sign, between each of the discharge coefficients given by the authors and the corresponding discharge coefficients according to the respective formulas.

Fig. 73 shows the average absolute values of the differences between the observations of Schoder and Turner and the corresponding μ_0-values of Formula (12) for very small heads between 0.015 m. and 0.025 m. on the different weirs. The difference for these small heads of nearly the same height is the arithmetic average of the μ_0-values. These differences are small and reach only 0.0023, or 0.33 per cent.

Fig. 74(*a*) shows the differences between the coefficients of discharge determined from the test data of Schoder and Turner, Schaffernak, Lindquist,

and the Swiss Department of Hydraulics and the corresponding coefficients calculated by Formula (12). The magnitude of the differences is indicated by the symbols shown in the legend (Fig. 74(*d*)). It is noteworthy that nearly 50% of all the differences are less than 0.0025 and that more than 75% are less than 0.0050. With one exception the differences which exceed 0.010 are only for heads less than 0.02 m.

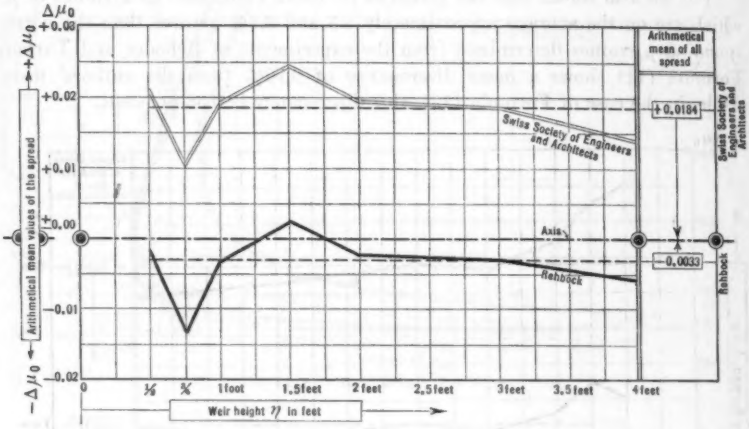
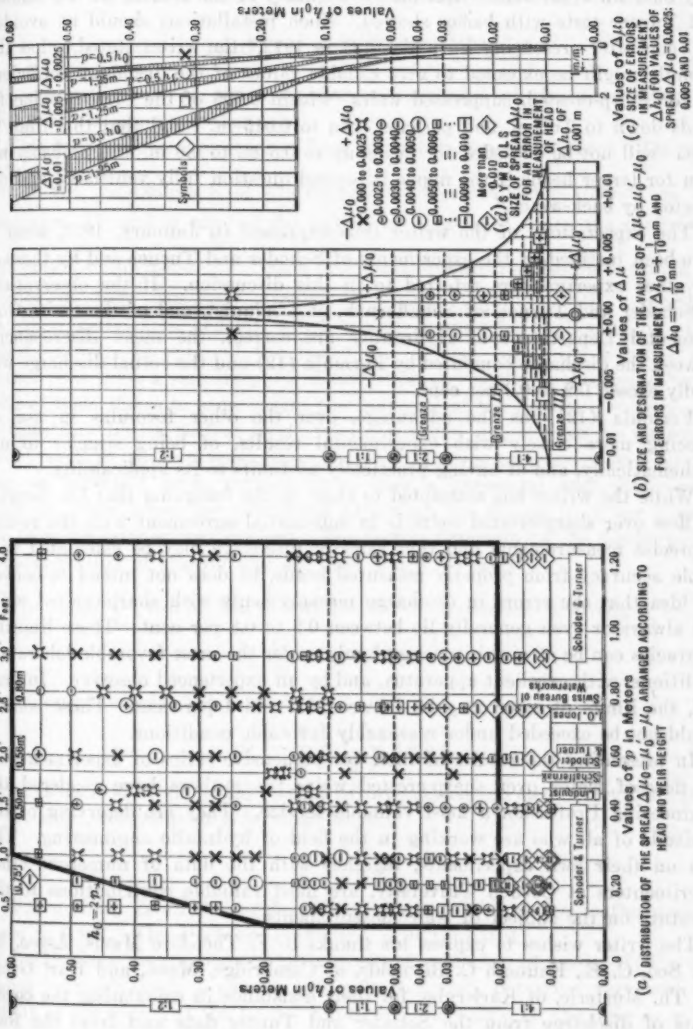


FIG. 73.—COMPARISON OF ARITHMETIC MEAN VALUES OF Δ μ_0

It will be seen that there are no larger zones in this diagram where the differences are consistently greater than 0.0050. In general, these relatively large differences are surrounded by smaller differences some of which are positive and some negative. This fact would lead one to the conclusion that the larger differences are due more likely to experimental errors than to inexactness of Formula (12). The heavy black line (Fig. 74(*a*)) indicates the field usually covered by weir measurements.

From Fig. 74(*b*) it is possible to determine the effect of an error of 0.0001 m. in measuring the value of the head, h_0, on the corresponding value of μ_0.

Slight irregularities in the magnitude of the velocity of approach to the weir as they occur in a normal channel do not have a material influence on the coefficient of discharge as long as the cross-section of the approach channel is the same throughout its entire length and as long as the flow is in an axial direction. Schaffernak conducted two series of experiments on weirs, one with and the other without screens in the approach channel and found that, although the velocity distribution was quite different in the two cases, the coefficients of discharge differed not more than by 0.2 per cent. The close agreement between coefficients of discharge as determined from the tests made in Amsteg, Karlsruhe, Ithaca, N. Y., Stockholm, and Vienna, each with different testing installations, would seem to indicate also that slight irregularities in the velocity of approach have little effect on the results. Large



(a) DISTRIBUTION OF THE SPREAD $\Delta\mu_0=\mu_0'-\mu_0$ ARRANGED ACCORDING TO HEAD AND WEIR HEIGHT

(b) SIZE AND DESIGNATION OF THE VALUES OF $\Delta\mu_0=\mu_0'-\mu_0$ FOR ERRORS IN MEASUREMENT $\Delta h_0=+\frac{1}{10}$ mm AND $\Delta h_0=-\frac{1}{10}$ mm

(c) Values of $\Delta\mu_0$ SIZE OF ERRORS IN MEASUREMENT Δh_0 FOR VALUES OF SPREAD $\Delta\mu_0$=0.0025, 0.005 AND 0.01

FIG. 74.—COMPARISON OF THE μ_0-VALUES.

variations in the magnitude of velocity and the direction of flow in the approach channel, however, as they can be produced by baffles which are improperly placed, or by abrupt changes in the section of the approach channel, may have an appreciable effect on the accuracy of the results, as the Schoder and Turner tests with baffles showed. Such installations should be avoided.

Soon after presenting Formula (12) in 1912,* the writer stated,† that this formula might be expected to give reliable values of the discharge coefficient, μ_0, for sharp-crested, suppressed weirs "within 0.005 of the correct value for heads down to 0.02 m. and perhaps even to 0.015 m.", and that this limit of 0.005 "will not be exceeded for heads up to 0.5 m. to 0.6 m. and probably not even for larger heads if the nappe is beyond question fully ventilated and not affected by back-water."

The expectations of the writer thus expressed in January, 1913, seem to have been justified by the experiments of Schoder and Turner and by those of the other experimenters referred to in this discussion. If the observations of Schoder and Turner, of Schaffernak, of Lindquist, and of the Swiss Governmental Department of Hydraulics are correct, the mean discrepancies between the discharge computed by Formula (12) and the actual discharge will hardly exceed 0.2 to 0.3 per cent.

Formula (12) has the advantage, over the other formulas in use, of agreeing more closely with experimental results, of being simpler to use mathematically, and of having practically no limits to its applicability.

While the writer has attempted to show in the foregoing that his formula for flow over sharp-crested weirs is in substantial agreement with the results of precise measurements and that rates of discharge may be calculated with ample accuracy from properly measured heads, he does not intend to convey the idea that the errors in discharge measurements with sharp-crested weirs will always or even generally lie between 0.2 to 0.3 per cent. These limiting accuracies can be attained in general only under the most favorable laboratory conditions, with excellent apparatus, and by an experienced observer. In general, the limits of error may lie between 0.5 and 1 per cent. These bounds should not be exceeded under reasonably favorable conditions.

In presenting the results of their comprehensive series of experiments on the flow of water over sharp-crested weirs the authors have rendered the Engineering Profession a most valuable service. They are deserving of the gratitude of all who are working in the field of hydraulic engineering. The data on their own experiments, together with the data of numerous other experimenters at Cornell University, are most valuable contributions to the literature on the subject of weir measurements.

The writer wishes to express his thanks to F. Theodore Mavis, Assoc. M. Am. Soc. C. E., Kenneth C. Reynolds, of Cambridge, Mass., and Karl Grein and Th. Musterle, of Karlsruhe, for their assistance in calculating the coefficients of discharge from the Schoder and Turner data and from the four formulas quoted, and to Professor Mavis for his translation of this discussion from the German.

* Handbuch der Ingenieurwissenschaften, dritter Teil, 2. Band, 1. Abteilung, Leipzig 1912, p. 58.

† *Zeitschrift* des Verbandes Deutscher Arch. u. Ing. Vereine, 1913, Nr. 1.

ERIK G. W. LINDQUIST,* M. AM. SOC. C. E. (by letter).—This interesting and valuable paper shows that the distribution of velocity in the channel on the up-stream side of a sharp-crested weir has a great influence on the discharge. On the basis of their experiments, the authors have derived a formula,

$$Q = 3.33\ L \left[\left(h + \frac{v_a^2}{2g} \right)^{\frac{3}{2}} + \frac{h}{3.33} \times \frac{v_b^2}{2g} \right]$$

(*D*), the application of which requires a knowledge of the distribution of velocity in the approach channel.

This formula does not seem to have a correct theoretical basis. The first term in the parenthesis is of the dimension, $L^{\frac{3}{2}}$, while the second has the dimension, L^2, and is not "of the nature of a volume", since the coefficient, 3.33, has the dimension, $L^{\frac{1}{2}}$. Such dimensionally incorrect formulas should be avoided as they can only express the result obtained with a particular experimental arrangement, which usually cannot be duplicated by other experimenters. In spite of the fact that, in most cases, the Schoder and Turner tests show a very irregular distribution of velocity, a theoretical treatment of them shows to some extent the influence of the factors involved.

Consider, first, a perfect fluid (frictionless and not affected by molecular forces, such as surface tension) under the influence of gravity and flowing over an infinitely high, vertical weir, with a horizontal and sharp crest. Since no other forces are considered as acting, the stream lines are parallel and have the same energy content. The factors determining the discharge are the acceleration of gravity and the head above the crest.

The ratio, $\frac{Q}{L}$, can be found, with the aid of dimensional analysis, to be,

$$\frac{Q}{L} = \phi \sqrt{2g}\, h^{\frac{3}{2}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (20)$$

Combining this with the formula,

$$Q = \frac{2}{3} \mu L \sqrt{2g}\, h^{\frac{3}{2}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (21)$$

which is derived by the usual method of integrating the elementary discharges of the sheet, the value of ϕ is found to be,

$$\phi = \frac{2}{3} \mu \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (22)$$

For a sharp crest, the coefficient, ϕ, can be shown to be mathematically equal to 0.407. The discharge coefficient, μ, is thus 0.611.

Under these assumptions, μ has a constant value for all heads. If the weir is not infinitely high, μ can no longer be constant, but must vary with the height of the weir and the head. Indicating the height of the weir as w, the head as h, and the velocity head as k, μ must be a function of the ratios, $\frac{h}{w}$, $\frac{k}{h}$, or $\frac{h}{h+w}$. Theoretically, μ must be a constant when $w = \infty$, and

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equal to 1 when $h = \infty$, or when $w = 0$. From these considerations it is evident that it is not sufficient to assume that,

$$\mu = m + n\frac{h}{w} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (23)$$

since μ would then become ∞ when the height of the weir is very small in comparison with the head. The expression, $\frac{h}{h+w}$, satisfies the theoretical conditions.

For a perfect fluid, the velocity head is,

$$k = \frac{v^2}{2g} = \frac{Q^2}{L^2 (h+w)^2 2g} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (24)$$

and combining Equation (24) with Equations (20) and (21), the resulting ratio is,

$$\frac{k}{h} = \phi^2 \left(\frac{h}{h+w}\right)^2 = \frac{4}{9}\mu^2 \left(\frac{h}{h+w}\right)^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots (25)$$

Therefore,

$$\mu = f_1\left(\frac{k}{h}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (26)$$

Considering now a real fluid (viscous, adhesive, and having a certain surface tension) flowing over a weir of finite height, the energy content of the different stream lines is not the same, due to the viscosity. For this reason, two weirs having the same value of $\frac{h}{h+w}$ will show the same value of μ only when the distribution of velocity is the same at the sections at which the head is measured. If these distributions are determined by the viscosity alone, they can be similar only when the values of the Reynolds' number are the same. This criterion of similarity is the product of the mean velocity in the section, a length characterizing the section, and the reciprocal of the kinematic viscosity.

Thus,

$$E = \frac{v\ l}{\nu} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (27)$$

The length can be chosen as the hydraulic radius, $R = \frac{A}{P}$, giving,

$$E = \frac{1}{\nu}\frac{Q}{A}\frac{A}{P} = \frac{1}{\nu}\frac{Q}{P} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (28)$$

In this case, P, the wetted perimeter, is, $L + 2(h+w)$. Hence,

$$E = \frac{1}{\nu}\frac{Q}{L + 2(h+w)} = \frac{1}{\nu}\frac{Q}{L}\frac{1}{1 + 2\left(\frac{h+w}{L}\right)}$$

Among other things, the Reynolds' number is dependent on the width and height of the weir and the operating head. Consequently, μ must be a function of E and the complete expression is

$$\mu = F\left\{\frac{k}{h}, E\right\} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (29)$$

Experiments to determine the loss of head in smooth pipes and the resistance of thin smooth plates, towed through still water, have shown that the Reynolds' number enters the resulting equation as $f\left(\frac{1}{E}\right)$ rather than as $f(E)$, showing that at high speeds or large dimensions the viscosity does not affect the magnitude of these quantities. Hence, for practical cases, where L and w are usually large, the influence of the viscosity can be neglected. However, it must be stated that this conclusion only holds true when the type of flow is turbulent. For very low heads on high weirs, laminar flow may occur, causing a change in the conditions.

In most cases met in practice, the distribution of velocity in the cross-section at which the head is measured, is determined chiefly by other forces or arrangements than the viscosity and is not governed by any known mechanical law. To take into account the influence of the distribution of velocity on the discharge, the relation between the velocity heads as computed from the mean velocity may be compared with the mean kinetic energy of the individual stream lines or mass elements.

Considering a mass element, $d\,m$, flowing through a cross-sectional area, $d\,A$, with a velocity, v_y,

$$d\,m = \rho\,d\,A\,v_y \quad \ldots\ldots (30)$$

the velocity head is,

$$d\,k = \frac{\rho\,d\,A\,v_y\,v_y^2}{2\,g} \quad \ldots\ldots (31)$$

and for the whole cross-section, the kinetic energy is,

$$\frac{\rho}{2\,g}\int_0^A v_y^3\,d\,A \quad \ldots\ldots (32)$$

The mean velocity at the section is,

$$v_m = \frac{\int_0^A v_y\,d\,A}{A} \quad \ldots\ldots (33)$$

and the total mass flowing through the section is,

$$m = \rho\int_0^A v_y\,d\,A \quad \ldots\ldots (34)$$

Hence, the mean velocity head is,

$$k_m = \frac{\rho}{2\,g}\,\frac{\left[\int_0^A v_y\,d\,A\right]^3}{A^2} \quad \ldots\ldots (35)$$

The ratio between these differently computed velocity heads is,

$$\delta = \frac{A^2\int_0^A v_y^3\,d\,A}{\left[\int_0^A v_y\,d\,A\right]^3} \quad \ldots\ldots (36)$$

As δ is not a constant, but varies from case to case, a further condition arises, namely, that,

$$\mu = \phi\,\{\delta\} \quad \ldots\ldots (37)$$

Neglecting the influence of purely viscous flow, the relation thus far discovered is,

$$\mu = \psi\left[\frac{k}{h}, \delta\right] \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (38)$$

When the distribution of velocity is determined by the viscosity alone, δ must be a function of E.

In the Schoder and Turner experiments, the value of δ, computed on the basis of the curves for the vertical element at mid-stream, varies as follows:

Series	E, Run 53 to 57	1.07
"	I, Run 54 to 58	1.08
"	I, Fence 1	1.60
"	I, Fence 3	2.08
"	D, Run 101 to 105	1.80
"	D, L, M (Fig. 46)	2.0

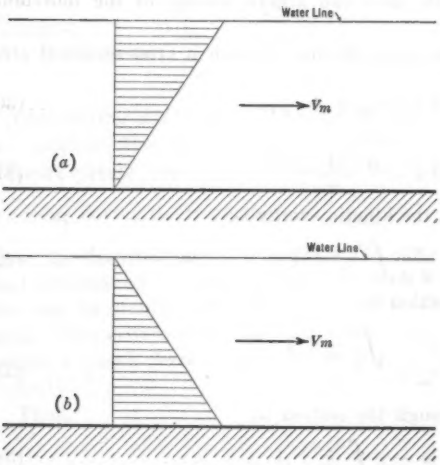


FIG. 75.

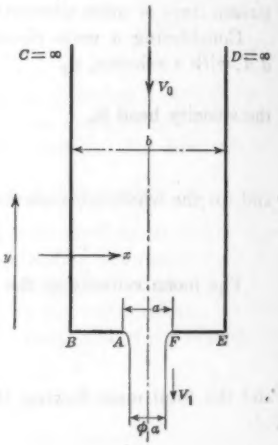


FIG. 76.

However, the question arises as to whether δ completely determines the influence of the velocity distribution on μ. Imagine, for instance, two distribution curves with the same value of δ and with the same mean velocity, but with different surface and bottom velocities, as shown in Fig. 75. It seems possible that the μ-values will differ. In order to find other factors determining the μ, consider another problem capable of being treated mathematically, namely, the discharge of a fluid through a very long rectangular orifice in the bottom of an infinitely high basin (Fig. 76).

Assume that the flow is the result of an external pressure, p_0, on the surface instead of the force of gravity. Under the assumption that viscosity is not affecting the flow, the stream lines are parallel at infinity and have the same energy content.

From Bernouilli's theorem,

$$p_0 + \rho \frac{v_0^2}{2} = p + \rho \frac{v^2}{2} \quad \ldots\ldots (39)$$

in which, p and v are the pressure and velocity at any point and ρ is the density of the fluid. At some point beyond the orifice, the stream lines are parallel and have the same velocity, v_1, which the outer stream lines have had since leaving the orifice, because the surface of the jet is under constant pressure. Hence,

$$p_0 + \rho \frac{v_0^2}{2} = \rho \frac{v_1^2}{2} \quad \ldots\ldots (40)$$

If q is the discharge per unit length and ϕ is the coefficient of contraction (see Fig. 76),

$$q = v_0 b = \phi a v_1 \quad \ldots\ldots (41)$$

In order to find an expression for computing ϕ apply the law according to which the change in momentum between the initial and final sections is equal to the forces acting. The change in momentum is $\rho q (v_1 - v_0)$, and the forces are due to the pressures, p_0, and the pressures on the walls. These latter pressures may be denoted by $\int p \, dx$ and $\int p \, dy$. The integration is to be made over the entire boundaries and the differentials have the following values:

		dx.	dy.
Stretch	A-B	—	0
"	B-C	0	+
"	D-E	0	—
"	E-F	—	0

Hence, from symmetry,

$$\rho q (v_1 - v_0) = p_0 b + \int_A^F p \, dx \quad \ldots\ldots (42)$$

Combining Equation (42) with Equations (39), (40), and (41), and simplifying, the result is,

$$\phi = \frac{1}{1 + \sqrt{1 - \frac{a}{b} + \frac{a}{q^2} \int_A^F v^2 \, dx}} \quad \ldots\ldots (43)$$

From Equation (43), it seems that the contraction coefficient depends on the velocity of flow along the walls approaching the orifice. The pressure on this face will decrease as the velocity increases. If the value of the integral is known it is possible to compute the value of the coefficient.

This analysis has been made by assuming an ideal fluid not acted upon by gravity. According to experiment, with gravity acting, the fluid filaments are sensibly parallel a short distance outside the orifice and, hence, this factor has only a slight effect on the path of flow. As for the motion of the fluid within the basin, it is immaterial whether the flow is assumed to take place under the action of gravity or a constant pressure.

The presence of fluid friction in a real fluid does not materially change any of the conclusions drawn from Equation (43).

If a vertical weir in an open channel is assumed to be one-half the basin just considered, with the height, $\frac{b-a}{2}$, and the head, $\frac{a}{2}$, it is necessary to measure the pressure on the face of the weir to determine the total effect of the velocity distribution on the value of μ. As such measurements have not previously been made in connection with measurements of the coefficient it is not possible to fix the total influence of the distribution.

The molecular forces may affect the discharge of a sharp-crested weir in one of three ways: First, either by adhesion, causing the nappe to cling to the crest, and giving the effect of its being rounded; second, by the surface tension acting on the surfaces of the sheet and giving a force in the same direction as the force of gravity, thereby causing an increase in the vertical acceleration of the discharge; and, third, by a combination of the other two.

If the rounding of the crest may be considered as constant for all heads, its influence will be greater at lower than at higher heads and the relation may be written,

$$\mu = f\left(\frac{\varepsilon}{h}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (44)$$

in which, ε is the radius of the crest caused by the adhesion. The effect of the surface tension may be taken into account by using the expression,

$$\mu = \chi\left(\frac{T}{\rho\, g\, h^2}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (45)$$

in which, T is the surface tension and $\rho\, g$ is the weight per unit volume. This function can be found by the aid of dimensional analysis.

The influence of the molecular forces may be illustrated by Figs. 77 and 78, which show the same discharge over a sharp-crested weir, 700 mm. high, made of brass. The writer's experiments have shown that there will be no discharge over such a weir when $h = 0.0026$ m. When $h = 0.0028$ m., the discharge begins. If this is slowly increased, the sheet will cling to the face until the head is 0.059 to 0.062 m., when it will become free. By touching the sheet, or by introducing air below it, it can be freed when $h > 0.0105$.

Within this range it is possible to make the sheet cling by means of a board, etc. By slowly lowering the head, the sheet will remain free until $h = 0.010$. It will begin to cling at some point below this head and is always clinging at $h = 0.0088$ m. At heads greater than 0.062 m., the sheet will be free. These values are for a water temperature of 11° cent.

The writer's studies have shown that the coefficient of discharge, μ, is a function of the dimensionless quantities, $\frac{k}{h}$, δ, $\frac{\varepsilon}{h}$, and $\frac{T}{\rho\, g\, h^2}$, and that it will have the same value when each of the ratios is the same simultaneously.

With respect to the conditions expressed in Equation (38) the writer has studied the function and the value of δ by plotting known experimental values in a diagram, with μ as the ordinates and $\left[\mu\frac{h}{h+w}\right]^2$ as the abscissas. For

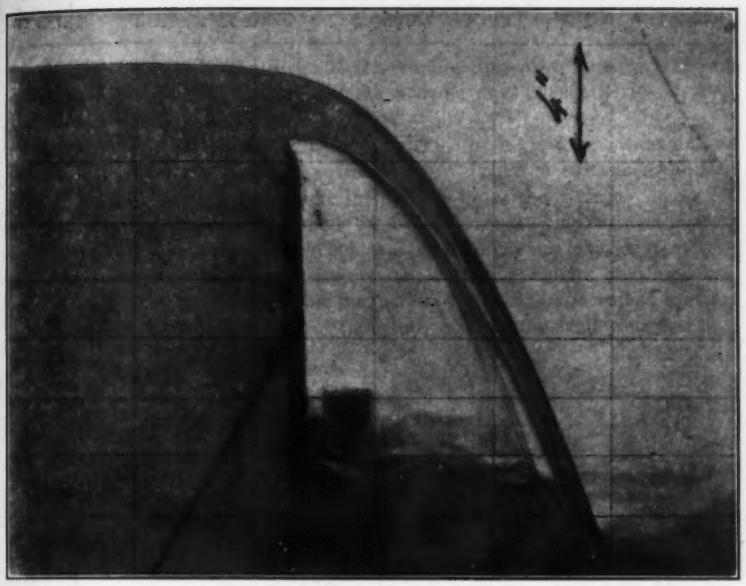


FIG. 77.—DISCHARGE OVER SHARP-CRESTED WEIR, $h > 0.0105$.

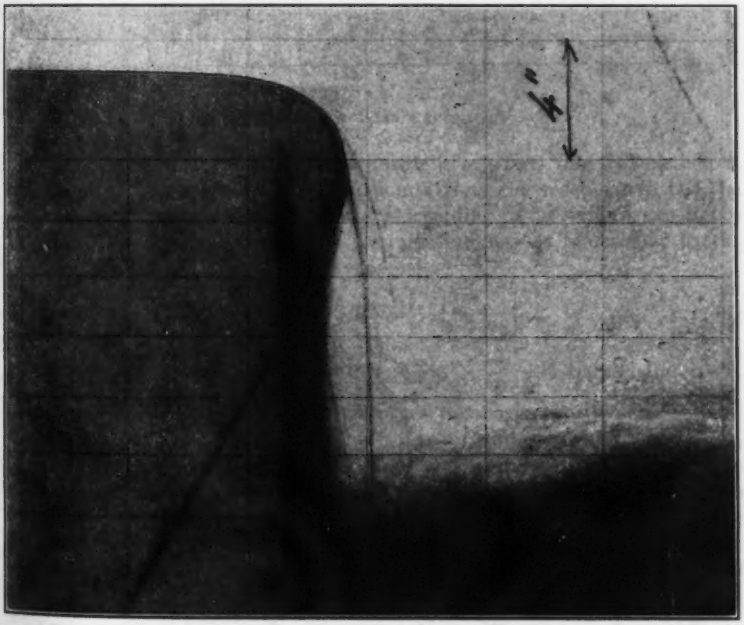
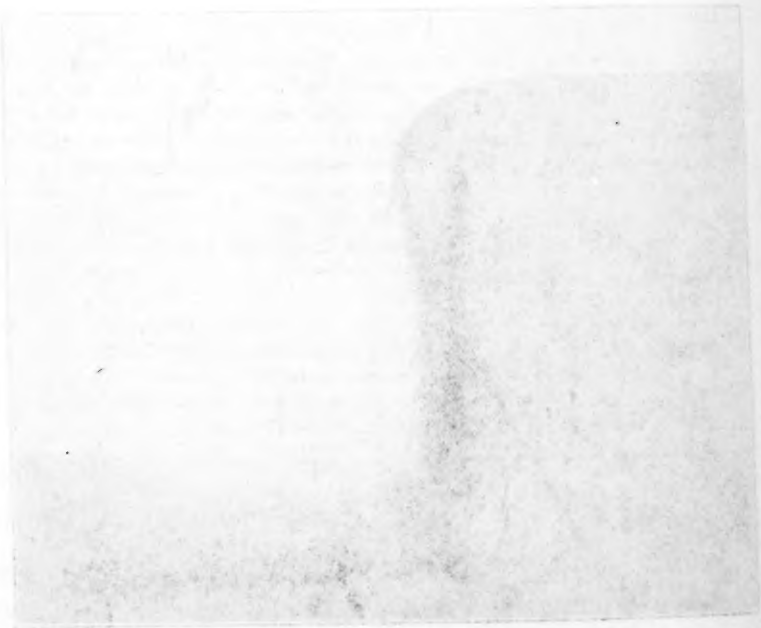


FIG. 78.—DISCHARGE OVER SHARP-CRESTED WEIR, $h < 0.0105$.



this purpose the tests by the Swiss Hydrographic Bureau at Amsteg,* Bazin's experiments Nos. 5, 7, 9, and 10, and the Schoder and Turner Series D to O, have been chosen.

In the experiments at Amsteg the weir was 0.8 m. high, 3.0 m. wide, and its distance from the draft-tube of the Amsteg plant was 23 m. Between the weir and the turbines thirteen racks were installed to quiet the surface.

The Bazin experiments were made in a channel 2 m. wide, using the same weir crest. The channel of approach was 44 and 95 m. long, respectively, measured below a calibrated, sharp-crested weir, the height of which was 1.135 m. The known fact that the Bazin tests give discharge values that are too high, does not affect the value of this study because the same weir was used as an indicator of the discharge.

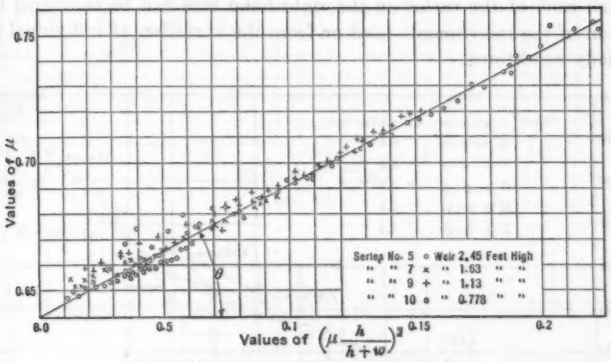


FIG. 79.—BAZIN'S WEIR EXPERIMENTS.

Fig. 79 shows that the Bazin experiments give points that fall close to a straight line. The divergence of the points from the mean curve may be attributed to the experimental error, which according to the writer's experience affects the third decimal place. The average line may be represented by the formula,

$$\mu = 0.640 + 0.515 \left[\mu \frac{h}{h + w}\right]^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots (46)$$

The computed value of δ is, therefore, $0.515 \times \frac{9}{4} = 1.16$. These results show that the distribution of velocity has been regular and fairly similar in all the experiments.

Before studying the Schoder and Turner tests in the same manner as those of Bazin, it is first necessary to examine the experiments on high weirs (Table 1), particularly the Series D, J, K, L, M, and O, respectively, as to the relation between values of μ, the head, and the distribution of velocity. Fig. 80 shows that these series give the same value of μ within the limits of the probable experimental error.

* Communications du Service des Eaux, No. 18, "Contribution à l'étude des méthodes de jaugeage".

The authors do not indicate the probable errors of the experiments in spite of the great assistance that such a computation gives for judging the reliability of experimental results. The formula for determining the error is obtained by taking the partial differential of Equation (21) which gives,

$$\frac{\delta Q}{Q} = \frac{\delta \mu}{\mu} + \frac{\delta L}{L} + \frac{3}{2}\frac{\delta h}{h} \quad \ldots\ldots\ldots\ldots\ldots\ldots (47)$$

Since,

$$Q = \frac{\text{Volume}}{\text{Time}} = \frac{V}{T}$$

the error is given by,

$$\pm \frac{\delta \mu}{\mu} = \pm \frac{\delta V}{V} \mp \frac{\delta T}{T} \mp \frac{\delta L}{L} \mp \frac{3}{2}\frac{\delta h}{h} \quad \ldots\ldots\ldots\ldots (48)$$

in which each of the ratios on the right-hand side can be estimated from the precision of the instruments used or from the variation of individual readings for steady conditions.

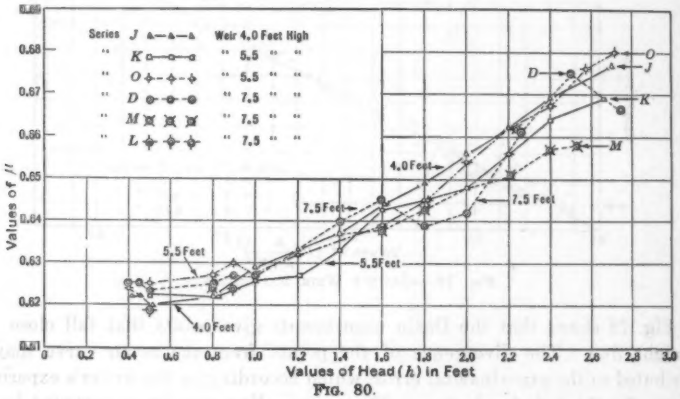


FIG. 80.

Comparing the distribution diagrams for the Series D, K, L, M, and O with those for Series J, it is found that, upward from a point 4 ft. below the crest, the diagrams are fairly similar and that below this point the depth has very little effect on the currents. From these experiments, it seems to the writer to be quite impossible to make a decision as to the applicability of any formula, since the results are so greatly influenced by the experimental arrangements used by the authors.

The Schoder and Turner Series E, F, and N are shown in Fig. 81. It will be noted that the points can be represented by a straight line, the slope of which is θ. In the lower range of the abscissas, the points diverge from this line by increasing amounts as the abscissas decrease. The three series of experiments give the formulas:

Series E and F,

$$\mu = 0.621 + 0.57\left[\mu \frac{h}{h+w}\right]^2; \ \delta = 1.28 \quad \ldots\ldots\ldots (49)$$

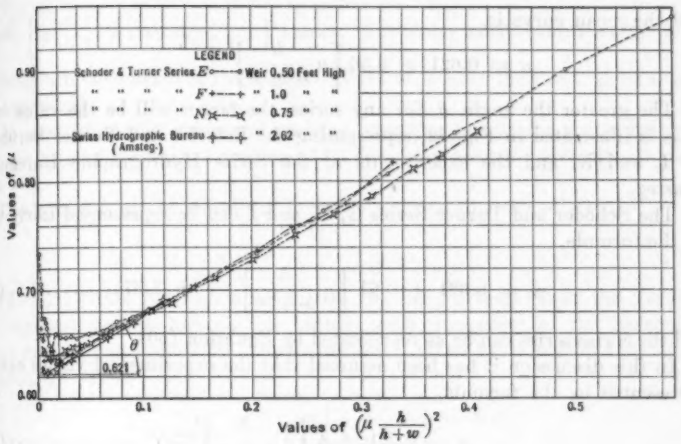

LEGEND
Schoder & Turner Series E Weir 0.50 Feet High
" " " " F " 1.0 " "
" " " " N " 0.75 " "
Swiss Hydrographic Bureau (Amsteg) " 2.62 " "
Values of μ
0.90
0.80
0.70
0.60
0
0.1
0.2
0.3
0.4
0.5
θ
0.621
Values of $\left(\mu \frac{h}{h+w}\right)^2$

FIG. 81.

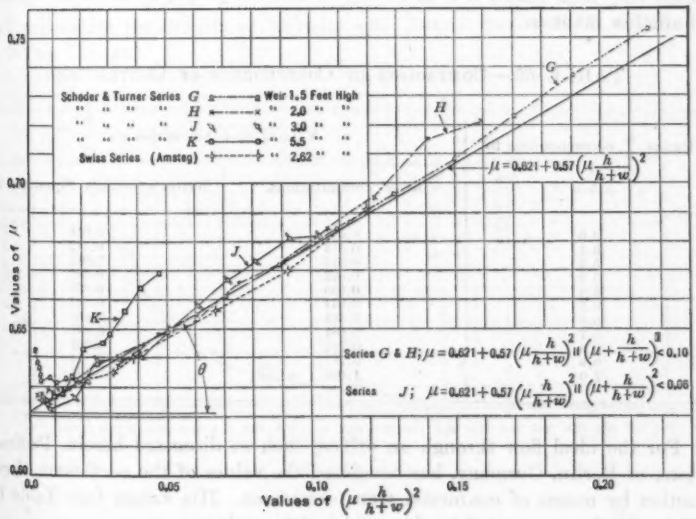

Schoder & Turner Series G Weir 1.5 Feet High
" " " " H " 2.0 " "
" " " " J " 3.0 " "
" " " " K " 5.5 " "
Swiss Series (Amsteg) " 2.62 " "
$\mu = 0.621 + 0.57\left(\mu \frac{h}{h+w}\right)^2$
Series G & H; $\mu = 0.621 + 0.57\left(\mu \frac{h}{h+w}\right)^2$ if $\left(\mu + \frac{h}{h+w}\right)^3 < 0.10$
Series J; $\mu = 0.621 + 0.57\left(\mu \frac{h}{h+w}\right)^2$ if $\left(\mu + \frac{h}{h+w}\right)^2 < 0.06$
Values of μ
0.75
0.70
0.65
0.60
0.0
0.05
0.10
0.15
0.20
θ
Values of $\left(\mu \frac{h}{h+w}\right)^2$

FIG. 82.

Series N,

$$\mu = 0.621 + 0.54\left[\mu\frac{h}{h+w}\right]^2;\ \delta = 1.21 \quad \text{.....(50)}$$

and the mean curve is,

$$\mu = 0.621 + 0.56\left[\mu\frac{h}{h+w}\right]^2$$

The greater the angle, θ, for any series, the larger will be the value of δ. This is illustrated in Fig. 82, representing the Schoder and Turner Series G, H, I, and K, and the experiments of the Swiss Hydrographic Bureau at Amsteg.

The Schoder and Turner Series G, H, and I can be represented fairly well by the formula,

$$\mu = 0.621 + 0.61\left[\mu\frac{h}{h+w}\right]^2;\ \delta = 1.37 \quad \text{.....(51)}$$

and the Swiss series can be as represented by Equation (50).

In this discussion it has been assumed that the experimental values can be represented by the formula,

$$\mu = 0.621 + \theta\left[\mu\frac{h}{h+w}\right]^2 \quad \text{.....(52)}$$

Equation (52) will give $\mu = 1$ for $w = 0$, only when $\theta = 0.379$; and it is, therefore, limited in its application. As there are no experiments on low weirs under very high heads, it is necessary to find the proper form of the equation in another manner.

TABLE 66.—Comparison of Coefficients of Contraction.

Ratios, $\frac{a}{b}$, corresponding to $\frac{h}{h+w}$	Values of ϕ According to:	
	Mises' computations.	Writer's formula, Equation (53).
0.0	0.611	0.611
0.1	0.612	0.612
0.2	0.616	0.616
0.3	0.622	0.622
0.4	0.633	0.633
0.5	0.644	0.645
0.6	0.662	0.663
0.7	0.687	0.687
0.8	0.722	0.721
0.9	0.781	0.774
1.0	1.000	0.941

For the ideal flow through an orifice, such as discussed herein, Professor Mises, of Berlin, Germany, has computed the values of the coefficient of contraction by means of conformal transformations. His values (see Table 66) can be closely represented by the empirical formula,

$$\phi = 0.611 + 0.33\left[\phi\frac{a}{b}\right]^2 \quad \text{.....(53)}$$

The agreement between the values found by Mises and those given by Equation (53) (which was derived on the assumption that the relation between ϕ and $\left[\phi \frac{a}{b}\right]^2$ is a straight line) shows, that under ideal conditions this type of formula is applicable if the highest values of ϕ near the limit, $\phi = 1$, are not considered.

On account of this agreement and the foregoing results the writer believes that the type of formula, Equation (52), for sharp-crested weirs, fits its purpose well.

The influence of the molecular forces is expressed by the relation,

$$\mu = \phi \left\{ \frac{\varepsilon}{h}, \frac{T}{\rho g h^2} \right\} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (54)$$

The form of the function is not known. As the μ diverges from the straight line representing the formula for higher heads, the effect of the molecular forces may be determined by subtracting the μ given by Equation (52) from the observed value.

$$\mu_{\text{Remainder}} = \mu - 0 \left[\mu \frac{h}{h + w}\right]^2 - 0.621$$

Plotting these values against the head gives, in foot-pounds,

$$\mu_{\text{Remainder}} = \frac{0.0007}{h^{\frac{3}{2}}}$$

which represents the results of Schoder and Turner Series E, F, G, H, J, and N (see Fig. 83).

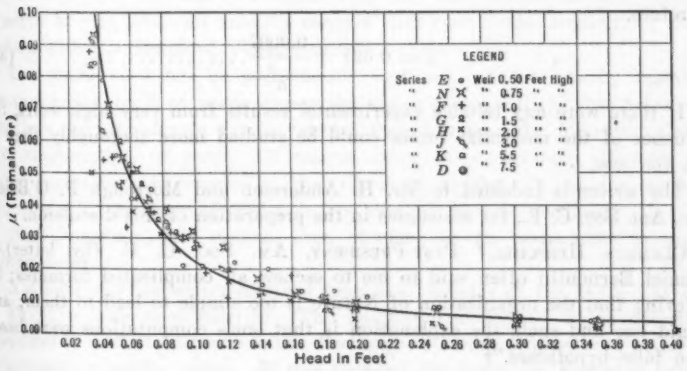


FIG. 83.

Since μ is a number, the numerator, 0.0007, has the dimensions, $l^{\frac{3}{2}}$, including the surface tension, T, which, in common with other factors, depends on the temperature.

On the basis of the Swiss, Bazin, and the Schoder and Turner tests, the formula for computing the discharge coefficient for a sharp-crested weir is,

$$\mu = 0.621 + \theta \left[\mu \frac{h}{h + w}\right]^2 + \frac{0.0007}{h^{\frac{3}{2}}} \quad \ldots\ldots\ldots\ldots\ldots (55)$$

or

$$\mu = \frac{1}{2\,\theta}\left(\frac{h + w}{h}\right)^2 \left\{1 - \sqrt{1 - 4\,\theta\left(0.621 + \frac{0.0007}{h^{\frac{3}{2}}}\right)\left(\frac{h}{h + w}\right)^2}\right\} \quad \ldots (56)$$

in which, θ has the following values,

Schoder and Turner Series E and F	0.57
" " " " N	0.54
" " " " G, H, and J	0.61
Swiss Series, Amsteg	0.54
(Bazin's Series Nos. 5, 7, 9, and 10	0.515)

The formula is rather complicated, but for practical use it can be represented graphically.

When the head on the weir is low, the velocity of approach has very little influence on the μ-value. Assuming, then, 0.001 as an upper limit for the error, ignoring the velocity head term and making $\theta = 0.55$ and $\mu = 0.62$, the resulting relation between the head and the height of the weir, at which the velocity of approach can be neglected, is:

$$\theta \left[\mu \frac{h}{h + w}\right]^2 < 0.001 \quad \ldots\ldots\ldots\ldots\ldots\ldots (57)$$

Hence,

$$\frac{h}{w} < 0.07$$

The formula for computing the μ-value at low heads, less than 0.07 w, is, therefore,

$$\mu = 0.621 + \frac{0.0007}{h^{\frac{3}{2}}} \quad \ldots\ldots\ldots\ldots\ldots\ldots (58)$$

If there were any reliable experimental results from very high weirs, the influence of the molecular forces could be studied more thoroughly than is now the case.

The writer is indebted to Mr. H. Andersson and Morrough P. O'Brien, Jun. Am. Soc. C. E., for assistance in the preparation of this discussion.

CLEMENS HERSCHEL,* PAST-PRESIDENT, AM. SOC. C. E. (by letter).—"Daniel Bernouilli often said to me to eschew all complicated formulas; he believing that the organization of Nature is too simple to lead to them; and should one find such, the explanation is that one's computations were based upon false hypotheses."†

"These things are beyond all use,
And I do fear them."‡

* Hydr. Engr., New York, N. Y.

† R. Wolf, "Biographies Relating to the History of Civilization in Switzerland," 1860, Vol. III, p. 176 (in French).

‡ Shakespeare, "Julius Caesar," Act II, Scene 2.

In reading the paper, and the learned discussions upon it, the question is likely to arise in the mind of a thoughtful hydraulic engineer:

"What is to be the outcome of so much labor, toward serving the profession in its work? Is there any use in it all? What is or should be a weir? Should it be the tool of an engineer, or is it a laboratory instrument, on which to found experiments and computations, and diagrams, and the laborious setting up of formulas to encompass these experiments? Should not an engineer, of all professions, constantly keep utility in view? And, surely, is not simplicity a measure of utility?"

Of course, every one knows the argument that science can not be fettered by considerations of utility; that science must be pursued for science's sake, and motives of utility must be rigorously banished; leading, as a possibly fine example, to the work of the German professor who spent a life time in translating the Latin classics into ancient Greek. But are such considerations properly presentable to engineers? Are not in these days, as many and more inventions and discoveries made by those who directly and specifically seek them, as by those who blindly grope and experiment, even though they do happen occasionally to lay the foundation for the utilitarian work of others?

Consider then, that a notable part of the work of a hydraulic engineer is to measure the flow of water, as he meets with it in his practice. This will include the flow of water in pipes, and in natural channels of a much larger size, such as rivers; and but little else. For the case of pipes a meter has been in existence since 1887, applicable alike to pipes ¼ in., or an unlimited number of feet (17 ft. actually in use) in diameter, the discharge formula of which is expressed in the simplest of terms, and is usually made to record its results automatically. The writer has lately seen a ponderous diagram proving once again that its record is correct, and in this particular case, that it coincides with the results found by the "salt velocity method"; or, as it might be put, "the salt velocity method" has been found accurate by comparison with the Standard Venturi meter.

The meter need not be permanently installed. A temporarily installed Venturi meter, built of wood, except the throat-piece, will in most cases cost less than the long drawn out operations made necessary by following almost any other method of work. Of course, there are exceptions, although not many, generally to be covered by the Gibson, or by the salt velocity method; but, as a rule, the sharp-crested weir has long ago become needlessly laborious in itself; needless, and useless, even obsolete, as an instrument to meter flowing water.

Not long ago the writer received from the Director of the Delta Barrage, in Egypt, a little book entitled, "The Discharge of Pipes", which described experiments made to determine the discharge of forms of pipe used in Egypt to measure irrigation water. It was founded, no doubt, on Egyptian habits, apparently in vogue since the time of the Pharaohs; but this little book, to the mind of the writer, is a good parallel to present-day dissertations on weirs and weir formulas. Their use seems to him to be as much out of date for metering water, as that of the forms of pipe used presumably for thousands of years past in Egypt, for the same purpose.

This is the age of machinery and mechanisms, as compared with ages of manual and mental work in the past. Offices are already filled with computing machines, typewriters, dictaphones, cameras, and what not besides; similarly, much mechanical work of the hydraulic engineer in the field can be replaced by the work of meters and their recording mechanisms. Take the case of the operation of the Aquedotto Pugliese, in Italy, which in its work of keeping what was nigh a desert, a flourishing district, uses 400 Venturi meters; or take New York's main water supply, that has three 17-ft. meters and more than seventy of smaller size.

To resume the consideration of weir formulas: Let any one look at the collection of such in the paper:

$$\mu_0 = 0.605 + \frac{1}{1\,020\,h_0 - 3} + 0.08\,\frac{h_0}{d_0}$$

for the value of one term to be used in computing according to the formula that contains it; and ask himself the question: Whether this is not the case of "a poor sport that is not worth the candle".

The defects of all weir gaugings are not new matters for the consideration of hydraulic engineers; and it is both startling and discouraging to find how the present paper and discussion have completely ignored what was said on the subject by Floyd A. Nagler, M. Am. Soc. C. E., in 1918* and by the writer in 1920,† and the reader will wonder at the dilatoriness with which progress is made in the practical advance of the art of the hydraulic engineer.

As regards the paper† last referred to, it must be remembered that the experiments were made in an hydraulic laboratory, whose use, including installation of apparatus and its removal, of necessity counted only twenty-three days. They consisted of devising a more rational form of weir than the classic sharp-edged weir; together with an attempt to measure "head on the weir" with "correction for velocity of approach" automatically compensated for; and resulted in finding a weir the formula of discharge of which was:

$$Q = 5.50d \text{ (English measures)}$$
$$Q = 1.675d \text{ (metric measures)}$$

The exactitude of this result has been questioned by a group of students in the University of California; but merely to have made an approach to such a result, is already a great advance in the art. Naturally, the apparatus used, the first of its kind, is capable of palpable improvements; and should meet with them at the hands of the next experimenters on the "hollow crest weir". This has been since May, 1920, repeatedly expressed to inquiring friends, and for future possible service an extract from one such letter follows:

"I have learned of the very great importance of having the piezometer holes in the crest tube bored precisely at a right, or a proposed angle to the '2:1 slope'.‡ And I regret to state that this direction of bore was not done

* "Verification of the Bazin Weir Formula by Hydro-Chemical Gaugings," *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 105.

† "An Improved Form of Weir for Gauging in Open Channels," *Transactions*, Am. Soc. Mech. Engrs., Vol. 42 (1920), p. 191; see, also, *Engineering News-Record*, June 24, 1920.

‡ *Transactions*, Am. Soc. Mech. Engrs., Vol. 42 (1920), No. 36, p. 205.

with a mechanical direction of the boring instrument, but made only with direction guided by hand and eye, in case of the weir I built in Boston. There were also too many of these piezometer holes, so that one cannot now tell which hole or holes prevailed, and the precise angle at which it or they were bored.

"I believe, were I in position to repeat the experiments, I would follow the 'original intention'* and have only one piezometer hole—certainly not more than three. This would surely lead to a hollow crest weir, having a straight line formula. Whether it would also prove the same, to have the piezometer hole, or holes, at right angles to the 2:1 slope, I cannot now affirm.

"Another point: Referring to Figs. 4 and 5,† you will notice the lack of any and all test, in the true sense of the word, of an idea; the idea being, to correct mechanically—by the way the observations are taken—for 'velocity of approach'. I still think this apparatus is good, and that the idea has value."

To summarize:

(1) As the late Frederic P. Stearns, Past-President, Am. Soc. C. E., expressed it,‡ "in the present state of the art the weir is not an accurate instrument for the measurement of water"; and no improvement in its use has been made, since that was written.

(2) The weir, and especially the sharp-crested weir, is, or should be, grossly obsolete so far as cost, convenience, and time-saving in its use to meter water are considered.

(3) Experiments upon its discharge, and setting up of involved formulas to portray with more or less precision their results will not remove these defects; nor are they of use to the practicing hydraulic engineer.

Ernest W. Schoder,§ M. Am. Soc. C. E. (by letter).—The discussions supply a kind of international symposium of opinion on the sharp-crested weir as a precise measuring device. The Engineering Profession is particularly indebted to those who have discussed the paper, have contributed valuable additional experimental data, have made original analyses of the data given in the paper, or who have made searching comparisons of old and new data. The discussions merit much intensive study and consideration, and the writer feels certain that due credit will be forthcoming as soon as the engineering public has had the opportunity of weighing the various arguments advanced.

It seems unfortunate that about 1890 a combination of circumstances started the idea that a precision for estimated weir discharge within 1% of the truth was readily obtainable. This idea has persisted to a surprising extent, due largely to the tendency of human nature to focus attention on and remember vividly the occasional close agreements while relegating to the "scrap pile of unexplained divergences" the numerous more scattered results, although the latter are often quite consistent among themselves. The writer heartily approves Mr. Parshall's comment that "much work remains to be done". Although the paper with the discussions is a contribution toward the desired end, the second conclusion‡ of the late Frederic P. Stearns, Past-

* *Transactions*, Am. Soc. Mech. Engrs., Vol. 42 (1920), No. 36, p. 205.
† *Loc. cit.*, pp. 208–209; see, also, Nos. 42–45, pp. 207–208.
‡ *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 172.
§ Prof. of Experimental Hydraulics, Cornell Univ., Ithaca, N. Y.

President, Am. Soc. C. E., to which Professor King has called attention, may well serve as a guide: "That the art should be developed so that the causes of the variation in results may be discovered and eliminated". Both Mr. Herschel and Professor King quote also the first conclusion* of Mr. Stearns: "That in the present state of the art the weir is not an accurate instrument for the measurement of water."

It will not be amiss to invite every one making use of weir measurements to study carefully some curves presented by Mr. Stearns† in discussing another paper. In his remarks he calls attention to the wide divergences between the results of the several experimenters, amounting to fully 5% at 0.5-ft. head. Figs. 27, 28, and 29, and Fig. 49, of the paper give similar information.

If the data by Professor Pardoe had been available to Mr. Stearns, or to the writers, an even wider range of results would be indicated. For all ten heights of weir, from $y = 0.1$ to $y = 2.3$ ft. (see Fig. 59), Professor Pardoe's experimental coefficients are markedly lower than those found by Rehbock's formula (which is itself the lowest on Mr. Stearns' Fig. 25†); for instance, at 0.2-ft. head the coefficients are from 10.9 to 1.4% lower, and at 0.7-ft. head, from 8.0 to 1.3% lower. (See Table 67.) The writers and the discussors have explained several causes of differences, but it still seems necessary to seek the full explanation. No single authority has enough data at his command to enable him to classify definitely the reasons for all these departures. It is time that some international co-operative researches are undertaken.

TABLE 67.—COMPARISON OF PARDOE'S COEFFICIENTS FOR HIS 2-FOOT WEIR WITHOUT END CONTRACTIONS,* WITH COEFFICIENTS BY REHBOCK'S FORMULA.

Height of weir, d_0, in feet.	FOR $h = 0.2$ FOOT.		FOR $h = 0.7$ FOOT (EXCEPT FIRST THREE).		
	Rehbock.	Pardoe.	h.	Rehbock.	Pardoe.
0.1	0.781	0.694	0.35	0.894	0.823
0.2	0.701	0.669	0.4	0.773	0.742
0.3	0.675	0.652	0.55	0.758	0.730
0.4	0.661	0.637	0.70	0.750	0.733
0.5	0.653	0.630	0.70	0.722	0.706
0.7	0.644	0.625	0.70	0.690	0.679
1.0	0.637	0.622	0.70	0.666	0.655
1.5	0.632	0.619	0.70	0.647	0.638
2.0	.....	.....	0.70	0.638	0.630
2.3	0.628	0.619	0.70	0.634	0.626

* See Fig. 59.

Professor Rehbock shows by Figs. 64 to 74 that his formula fits very closely nearly all the "regular" runs (those with fairly uniform distribution of velocities in the approach channel) of the writers' Series D to P on weirs 4 ft. and less in height and for heads of not more than 2 ft.; and that this is the case also for several other sets of data of more limited extent. Professor Rehbock has stated that:

"Slight irregularities in the magnitude of the velocity of approach to the weir as they occur in a normal channel do not have a material influence on the

* *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 172.
† *Loc. cit.*, p. 170.

coefficient of discharge as long as the cross-section of the approach channel is the same throughout its length and as long as the flow is in an axial direction."

This conclusion seems to be justified. Two important questions, however, are involved: (1) What magnitudes of irregularities are permissible for particular limits of precision? and (2), how can one be certain of approximately "normal" distribution without measuring the velocities?

True, in many laboratories, permanent fixed baffles can be set by trial so as to give satisfactorily uniform distributions of velocities throughout a considerable range of heads. For such conditions an easily applied formula like Rehbock's (after being thoroughly tested) is obviously preferable to one involving additional measurements and more extended computations, such as in the writers' Formula (*D*). However, there is something to be said against an idolatrous acceptance of formulas of the rigid type like those of Bazin and Rehbock. Even in a laboratory, if the discharge through carefully adjusted baffles is increased beyond the ordinary operating range (and what experienced experimenter has not desired and striven at times to extend the usual limit?), the nature of the distribution of velocities in the approach channel is likely to change more or less abruptly. Then, one must either undertake re-adjustment of the baffles to secure, by laborious trials, a proper distribution for the new higher heads, or one must abandon the non-conforming formula for one more adaptable.

Moreover, the principles involved are not limited to a refined laboratory weir. In some cases for temporary weirs used in field or plant tests, and usually for dams in rivers, it may be out of the question to exercise much control over the channel of approach. If the bottom of the channel is sloping or otherwise irregular; if the pond becomes filled with gravel, sand, or silt; if the contractor has left (as is quite common) part of his coffer-dam to be submerged and forgotten (except by the approaching water); if a water-logged boat or any sort of jetsam prevents normal bottom velocities; if an ice cover or high winds up streamward retard normal surface velocities; if a flood of unprecedented magnitude appears on the scene; in such cases, the engineer, who desires the most accurate estimates possible, will make all the necessary measurements to portray conditions as they are, and then will select or devise a formula to suit. The writer grants that, for the utmost precision, it is desirable to control the alignment of the sides and bottom up stream and that it is necessary to prevent major cross-currents. Nevertheless, there arise many cases in which expediency demands that the conditions be accepted considerably short of the ideal. If doubt exists as to the distribution of velocities, measurements must be made and, when made, the data for Formulas (*B*) or (*D*) are available, practically irrespective of any extremes that may be found.

An interesting illustration is at hand. Professor Rehbock comments on the irregularity of the coefficients for the writers' 5.5 and 7.5-ft. weirs, in that the coefficients in instances are higher for the same head than for the 4-ft. weir. Then, after agreeing with the writers in the explanation that the obstruction caused by the deflector hood over the 30-in. supply pipe retards the velocities that would occur near the bottom of a normal channel, thus reducing the

"effective" height of the weir to a figure less than the apparent height, Professor Rehbock characterizes the data for the 5.5 and 7.5-ft. weirs (Series D, K, L, M, and O, Table 1) as "obviously unreliable". Of course, he means "unreliable" for a formula involving the height of the weir. The writer wonders what Professor Rehbock would do if forced to deal with cases in which the effective height is less or greater than the measured height. As for the statement that sharp-crested weirs more than 4 ft. in height are almost never used in practice, that certainly does not apply to many cases on record in engineering literature. At Cornell University sharp-crested weirs that are much higher have been and are being used.

Next, Professor Rehbock questions the accuracy of the data for heads greater than $1\frac{3}{4}$ ft. because there are disturbances to the law of similarity, in that the coefficients of discharge increase rather suddenly if the head is greater than about 2 ft. It requires only a short examination of the distributions of velocities as shown in Figs. 39, 40, 44, and 45, for one to become convinced that the ratio of the velocity above the crest level to the mean velocity was greater for the highest heads than for the medium heads. In short, the law of similarity required some modification to suit the facts of the case. The explanation ventured by Professor Rehbock that possibly the ventilation under the overfalling nappe was insufficient, does not apply to the writers' apparatus. The atmosphere had free access to the space beneath the nappe.

It seems to the writer that the real reason why a formula, such as Equation (14), may have to be limited to heads less than 2 ft. and heights of weir less than 4 ft. lies in the probable insufficient length of channel of approach for cases outside these dimensions. The law of similarity, in its simplest geometric demands, would require a much longer channel for high heads than for low heads (for a fixed height of weir); and, also, a longer channel for a high weir than for a low weir (for a fixed head). Strictly, the requirements of axial flow and of fairly normal distribution of velocities should be met at a cross-section far up stream from the weir, as, for example, a distance of about six to ten times the total depth of water. For a head of 2 ft. on a weir 4 ft. high, making a total depth of 6 ft., perhaps a length of 36 ft. would satisfy most requirements; but a head of 4 ft. on a weir 6 ft. high would then require a length of 60 ft., subject to a sufficiently effective system of baffles for the greater velocities.

The apparatus used by the writers obviously falls short of such requirements, but the data are not, therefore, lacking either in accuracy or reliability. The measurements of heads and discharges apply, not to idealized conditions, but to the particular velocity conditions placed on record in the paper. The writers purposely went to extremes in the matter of causing abnormally high and low velocities both near the surface and near the bottom. None of the discussors has questioned the marked effect of these abnormally non-uniform distributions, and, apparently, the writers' conclusion still stands, that "no visual inspection could distinguish between discharges that differ as much as 4 or 5% at the same head due solely to different distributions of velocities in the channel of approach."

The experiments described by the writers receive the approval of Mr. Streiff. Comparing them with those of Professor Rehbock, he states:

"Professor Rehbock's experiments should be placed in a different class. These [Rehbock's] are 'indoor' experiments, conducted on such a small scale that this latter circumstance may easily introduce results that are 1% or 2% too small".

In this connection it is interesting to abstract parts of a communication received from Professor Rehbock in April, 1928. He stated that the latest measurements made by the writers, as well as by the Swiss Government Department of Hydraulics and by Professor Lindquist, do not support any contention that the discharges computed by the Rehbock formula are 1% or 2% too small.

Professor Lindquist made 10 measurements which, on the average, were only 0.01% greater than that computed by Equation (14), and, for 15 Swiss measurements, the average was 0.2% greater. Furthermore, he states that the average for the 137 measurements made by the writers for normal approach, for heights of weir between 0.5 ft. and 4 ft. and for overflow heights between 0.8 in. and 2 ft., gives a value that is 0.16% less than the average as determined by Equation (14). Therefore, the values determined by use of this Rehbock equation are considered to be slightly greater than the average of measurements made by the writers rather than 1% or 2% less; that is, for weirs of "ordinary heights". According to Professor Rehbock, values for such unusual weir heights as 5.5 ft. or 7.5 ft. must be eliminated from the comparison because the crest of the weir was only about 5 ft. above the floor which was formed by the top of the deflector hood about 13 ft. up stream from the place where the head was measured in the writers' experiments. The use of such relatively great weir heights, according to Professor Rehbock, gave discharge values by Equation (14) that were much too small. He has been led to conclude from the latest measurements, previously mentioned, that his Equation (12) gives values for μ that are correct within a few thousandths.

Mr. Streiff states further that Table 59 "shows the tendency of the Rehbock formula to be on the small side and the Swiss 'S. I. A. V.' ('Schweizerische Ingenieur und Architekten Verein'), formula is shown to check more closely than that of Rehbock". In his letter, Professor Rehbock agrees that the measurements made at Amsteg are greater, but only by a very small amount which averages 0.2% more than the values determined by Equation (14). He states, however, that values computed by Equation (14) agree better with the Amsteg measurements than those computed by the formula of the Swiss Society of Engineers and Architects. He submits Table 68 to support this statement. It shows that the Rehbock differences in Column (8) are less than the S. I. A. V. differences (Column (6)), in twelve of the fifteen cases listed. In the mean value of the deviations listed, the S. I. A. V. mean is 1.7 times as great as the Rehbock mean.

From these and other observations, Professor Rehbock concludes that, despite occasional statements to the contrary, the weir is a well-qualified instrument for measuring the flow of water accurately. This view, he states, has been corroborated by numerous recent calibrations of sharp-crested weirs,

especially those of Professor H. Engels, at Dresden, Germany, J. M. Voith, at Heidenheim, Germany, Professor Lindquist, at Stockholm, Sweden, Professor Schaffernack, at Vienna, Austria, The Swiss Federal Department of Hydraulics, at Berne, Switzerland, Engineer J. Th. Thysse, at Delft, Holland, and the experiments conducted by the writers at Ithaca, N. Y. Experiments by all these authorities give results which Professor Rehbock declares check within a few thousandths of the values computed by Equation (12). In brief, Professor Rehbock states that,

"All these newer measurements show convincingly that sharp-crested suppressed weirs, with aerated jets, are exceedingly accurate devices for measuring flowing water, if only a sufficient aeration of the jet is allowed, if the free approach of the water is not disturbed by built-in structures, if the overflow height is determined with sufficient accuracy, and if an accurate formula for computing the discharge from the width of weir and height of overflow, is used."

TABLE 68.—Comparison of Rehbock Values with Measurements at Amsteg, Switzerland.

No.	Height of overflow, h_0, in meters.	Observation in Amsteg, μ_A.	Formula of the S. I. A. V., 1926, μ_S.	Rehbock formula, 1912, μ_R.	$\Delta\mu_S = \mu_A - \mu_S$.		$\Delta\mu_R = \mu_A - \mu_R$.	
					Absolute.	Percentage.	Absolute.	Percentage.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
1	0.10	0.6260	0.6248	0.6248	+0.0012	+0.19	+0.0012	+0.19
2	0.15	0.6269	0.6268	0.6265	+0.0001	+0.02	+0.0004	+0.06
3	0.20	0.6294	0.6305	0.6298	−0.0011	−0.18	−0.0004	−0.07
4	0.25	0.6323	0.6349	0.6338	−0.0026	−0.41	−0.0015	−0.24
5	0.30	0.6376	0.6401	0.6382	−0.0025	−0.39	−0.0006	−0.10
6	0.35	0.6428	0.6453	0.6427	−0.0025	−0.39	+0.0001	+0.01
7	0.40	0.6489	0.6508	0.6474	−0.0019	−0.29	+0.0015	+0.23
8	0.45	0.6542	0.6564	0.6521	−0.0022	−0.34	+0.0021	+0.33
9	0.50	0.6596	0.6618	0.6569	−0.0022	−0.33	+0.0027	+0.40
10	0.55	0.6646	0.6672	0.6617	−0.0026	−0.39	+0.0029	+0.43
11	0.60	0.6694	0.6726	0.6666	−0.0032	−0.48	+0.0028	+0.42
12	0.65	0.6741	0.6778	0.6715	−0.0037	−0.55	+0.0026	+0.39
13	0.70	0.6787	0.6830	0.6764	−0.0043	−0.63	+0.0023	+0.34
14	0.75	0.6832	0.6879	0.6813	−0.0047	−0.69	+0.0019	+0.29
15	0.80	0.6877	0.6927	0.6862	−0.0050	−0.73	+0.0015	+0.22
Arithmetic mean					−0.0025	−0.37	+0.0013	+0.20
Mean deviation					0.0027	0.40	0.0016	0.25

It seems to the writer that this statement of the chief features of a high-class measuring weir leaves too much uncertainty as to what is meant by the terms, "free approach" and "sharp-crested", and that it falls short in failing to mention the proved effect of variations in the roughness of the up-stream face of the weir near the crest.

It is obvious that Professor Rehbock depends on strict geometric and hydraulic similarity when he extends the formula derived from his small-scale measurements to much greater heights of weir and heads, expecting estimates of discharge within less than 1 per cent. Consequently, for such pre-

cision, it seems highly important to the writer that every influence known to affect the discharge even a small fraction of 1% should be pointed out clearly.

Any engineer called upon to make a precise estimate of discharge by weir measurement (having first to construct his weir), should recognize the magnitude of the percentage effects corresponding with various departures as, for example, from strict smoothness of the up-stream face of the weir plate, strict sharpness of the square up-stream edge of the crest, and from "normal" distribution of velocities in the channel of approach. To illustrate, if the head is 1 ft., and use is made of a formula like that of Professor Rehbock which assumes a fairly uniform distribution of velocities with, say, 2.1 ft. per sec. for mean velocity above the crest level, when actually it is 2.0 ft. per sec., then an error of almost 1% in estimated discharge will result.

Professor Rehbock exhibits a remarkably close agreement with his formula by several experimenters. By inference it would seem to be true that good systems of baffles (in rectangular channels with fairly smooth sides and bottom) cause nearly similar distributions of velocities. Nevertheless, the writer thinks that engineers will not be freed entirely from scruples until ample direct evidence of similarity of velocity conditions is submitted. It is pertinent to point out again that Professor Rehbock omitted the data of the writer's highest heads in making his comparisons. The writer feels that Mr. Streiff is justified (even if not because of the extension from small-scale experiments to large-scale predictions) in his feeling that some "circumstance may easily introduce results that are 1% to 2% too small" (or too large). In brief, it must be known positively that both the construction of the weir and the condition of the approaching water are similar before one can expect agreements within 1 per cent.

Professor Nagler presents experimental evidence in Table 58 to the effect that the ratio of ϵ to h was the same (0.108), for his own sharp crest and for Bazin's corresponding weir (by interpolation). No evidence is presented showing the effects on this ratio of various degrees of rounding the edge of the crest, but the effect of increasing the roughness of the up-stream face of the weir is shown to decrease the ratio. It does not seem at all conclusive to the writer that the agreement in ratios proves that Bazin's crest piece had a strictly sharp edge, or that the up-stream face was similar in smoothness to the "standard sharp crest" used by Professor Nagler. The question of the distribution of velocities arises, whether it was the same in Bazin's "standard weir", 3.72 ft. high, and in his comparison weir, 1.15 ft. high, located in his down-stream canal. The value of 0.112 for $\frac{\varepsilon}{h}$ was obtained from the weir 3.72 ft. high and the value of 0.099 from the weir 1.15 ft. high. Obviously, the matter of height of weir is a factor, and if there was a different distribution of velocities in the two cases the corresponding same-basis "effective heights" would not agree with the actual heights. Moreover, there does not seem to be any proved basis for using arithmetical interpolation to obtain the ratio for a weir 3 ft. high. The velocity conditions for Professor Nagler's 3-ft. weir, in fact the discharge itself, may have been sensibly different than for the cor-

responding height under Bazin's conditions. Everything considered, it would seem that many more data are needed to make this ingenious argument valid.

It is interesting to note that Professor Nagler has found increases in discharge due to roughening the up-stream face of the weir plate. Direct comparisons with Barr's data on V-notches and Weber's data with the roughening caused by grooving the weir plate are difficult to make, but the main fact seems to be well established. Coating the up-stream face with sand doubtless causes a condition like that of tuberculation on iron or lime deposits on brass; and grooving somewhat resembles pitting. In January, 1929, the writer made careful measurements of the up-stream water level and of the discharge over a crest piece made of a steel angle that had become badly tuberculated both on the up-stream face and on top. Without disturbing the inflow, the tuberculation was then cleaned off. It was a matter of considerable surprise to find that the up-stream water level did not change perceptibly, but it was reasoned that Nature had caused a balance. With the rust on top removed the same water level meant a greater head, but with the up-stream face cleaned this head was necessary to offset the increased contraction due to removal of the roughening of the up-stream face and the accompanying rounding of the up-stream top edge.

Inasmuch as Professor Nagler's hydro-chemical measurements have been mentioned in several of the discussions, and because Professor Rehbock has shown that his formula fits the writers' data very closely for reasonably uniform distributions of velocity in the channel of approach, Fig. 84 has been prepared* to show the distribution of velocities in mid-channel 25 ft. up-stream from Professor Nagler's weir. The surface velocities are low probably because of the stilling-raft floating on the surface a few feet up stream. Assuming that this represents also the distribution 16.35 ft. from the weir (where the head was measured), the mean values of $\frac{v_a^2}{2g}$ and $\frac{v_b^2}{2g}$ are obtainable for use in Formula (D), which gives (after adjusting v_a and v_b to agree with the first approximation to Q): for $h = 2.97$ ft., $Q = 18.315$ cu. ft. per sec. per ft., or 120.1 cu. ft. per sec. for the 2-m. (6.556-ft.) length. Rehbock's formula,

$$Q = \left[0.605 + \frac{1}{320\,h - 3} + \frac{0.08\,h}{d_0}\right] \frac{2}{3}\, L\, h \sqrt{2\,g\,h}$$

with $h = 2.97$ and $d_0 = 3.72$ ft., gives $Q = 18.307$ cu. ft. per sec. per ft., which differs only 0.13% from the value obtained by Formula (D). There is no other record now available to the writer for the distributions of velocities in Professor Nagler's channel of approach during his hydro-chemical gaugings, and, therefore, Formula (D) cannot be used for other heads.

Fig. 85 shows the lines from Professor Nagler's Fig. 24† with indicated discharges for heads of 2, 2.97, and 4 ft. by the formulas of Fteley and Stearns, Rehbock, and the writers' Formula (D), the latter for 2.97 ft. only. It appears that the hydro-chemical gaugings show fully 1% greater discharge than Rehbock's formula at 2, 3, and 4 ft.; they show the same as compared with

* *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 125, Fig. 11.
† *Loc. cit.*, p. 152.

the writers' Formula (D) at 2.97 ft.; and about 1% greater than the mean of Bazin's experiments at 2-ft. head. The latter coincides practically with the Fteley and Stearns formula,

$$Q = L\left[3.31\left(h + 1.5\,\frac{v^2}{2g}\right)^{\frac{3}{2}} + 0.007\right]$$

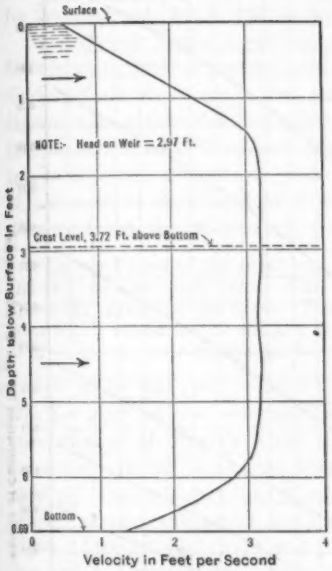


FIG. 84.

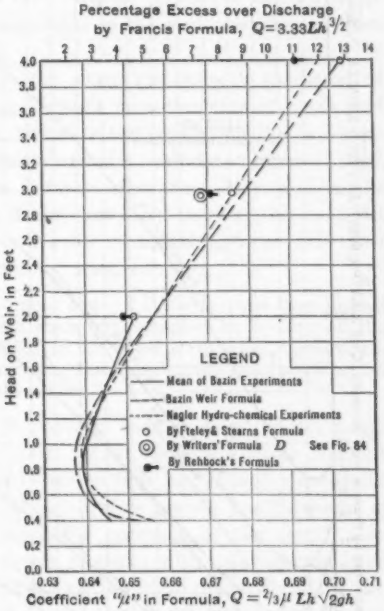


FIG. 85.

Concerning the cross-over trend of the values by the latter formula, it may be pointed out that Fig. 14 indicates the tendency of this formula at the greater heads to over-estimate discharges for weirs more than 3 ft. high, with velocity distributions as shown in Figs. 39 and 40, which distributions resemble approximately those made by Fteley and Stearns. These distributions are of the average type for flow in open channels, with near-surface velocities greatest, but only slightly greater than mid-depth velocities, and with near-bottom velocities least. Evidently, for a case with distribution more nearly uniform, as shown in Fig. 84, the discharge would be less, and Fteley and Stearns' formula would over-estimate to a greater extent. The 3.31 instead of the Francis 3.33 tends to keep the discharges low at the lower heads, but the 1.5 multiplier for the added term more than offsets this as the heads increase.

Bazin himself was very careful to state that the table of coefficients obtained from the averaging lines for his experiments was his own preference. The extension of his approximating formula far beyond the range of his experiments was never sanctioned by him, as far as the writer is aware. In order to show clearly what liberties have been taken with the so-called Bazin

"formula", Fig. 86 has been prepared. This diagram includes only the range of heads covered in the writers in Series A to P. The location, curvature, and slopes of lines should be compared with those in Fig. 9. It is true that

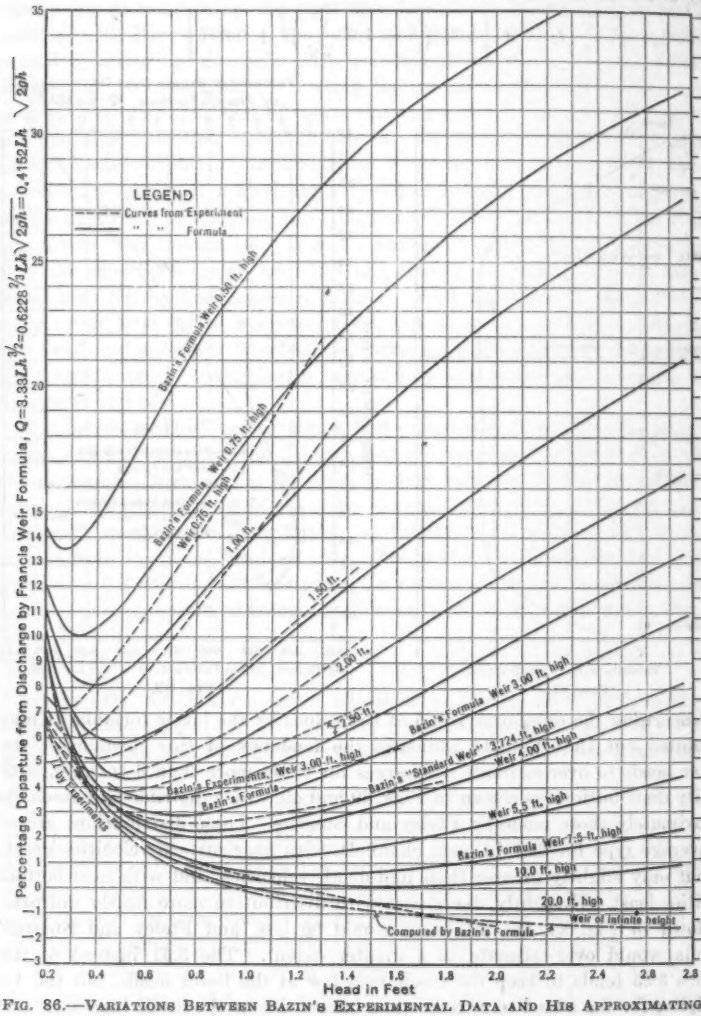


FIG. 86.—VARIATIONS BETWEEN BAZIN'S EXPERIMENTAL DATA AND HIS APPROXIMATING WEIR

FORMULA: $\left(Q=\left(0.405+\frac{0.00984}{h}\right)\left[1+0.55\left(\frac{h}{p+h}\right)^2\right]Lh\sqrt{2gh}.\right)$

heights and heads considerably beyond the range of Bazin's experiments are shown for the "formula", but a table* that has had wide circulation gives dis-

* *Water Supply and Irrigation Paper No. 200*, U. S. Geological Survey, pp. 187–189, Table 9.

charges computed according to the Bazin formula for heads up to 10 ft., with heights of weir from 2 to 30 ft. It is clearly seen that Bazin's approximating weir formula is a poor fit for his experiments, even within their range. For heads greater than the heights of weir the formula lines are concave downward, whereas the experimental data lines are concave upward throughout. The percentage extent of departures of the "formula" from the indications of Bazin's experimental data must be considerable in the region where the lines for the same weir are diverging rapidly.

If Professor Nagler's opinion as to the sharp and smooth condition of Bazin's weir plates is correct, and if the latter's measurements of heads and discharges are also correct, then one is forced to the conclusion that a future experimenter will be able to duplicate Bazin's results. All data presented since 1912, except those of Professor Nagler, indicate otherwise. Moreover, in the translated words of M. Hegly, who was Bazin's assistant, there are no clean-cut phrases such as those used by Professor Nagler in describing* the weir crest piece used with the Michigan hydro-chemical tests, namely,

"The back face of the plate and the upper edge of the brass was then planed and finished absolutely true. This left a sharp right-angled knife-edge on the up-stream corner of the crest. The object of the brass strip was to produce a crest which would not be subject to corrosion, thus preventing the introduction of error in the weir discharge from this source."

The writer has been assured that the words, "bien dressée" do not imply that the plate of iron was machine-planed. If, to quote Professor Nagler's translation of M. Hegly's letter: "From time to time he [Bazin] cleaned the top and up-stream face of the weir plate in order to remove small deposits of algæ and rust which might have affected the accuracy of the crest", then Bazin did what the writer has done scores of times with iron or steel crest pieces. Algæ intensify rust, and rust means pitting of an originally plane surface and dulling of an originally sharp, square corner. A few weeks of intermittent submergence in warm weather has a decided effect on ordinary rolled iron or steel.

Attention is called to the fact that for the runs with the very highest heads the writers found so much leakage into the stand-pipe through the hinge of the diverter that a perceptible volume of water was in mid-air, descending in the stand-pipe which was used as a measuring tank, and, therefore, did not appear on the float-gauge reading for the initial levels. The vertical distance between the initial and final levels in the tank was about 40 to 45 ft. Auxiliary tests, made to determine the rate at which this leakage water fell down the empty stand-pipe, and the lag (if any) in response by the float-gauge at "nearly full" and "nearly empty" levels, supplied the necessary data for estimating the effect. This condition was ignored in the computations leading to Tables 9 to 24, and, therefore, the tabulated discharges for heads greater than 2 ft. should be decreased by about 0.1% for a head of 2.25 ft., and 0.2% for a head of 2.5 ft. The effect on the lines at the right end of Fig. 18 is scarcely noticeable.

* *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 114.

The writer believes that he fully appreciates the comments that Mr. Herschel makes in discussing this paper. Whether the writers have done much toward finding the perfect weir may well be questioned. They had to meet a practical situation in which sharp-crested weirs, or what passed for such, met their immediate need. The research described in the paper was an offshoot from their main undertaking. Mr. Herschel was assured by the writer in correspondence of 1904 that other forms of weirs and other means of measuring the head difference were prominent features in his program for future experiments. That is still true, but the words of Edward Gibbon in his preface to the first volume of "The Decline and Fall of the Roman Empire" seem to apply: "The execution of the extensive plan * * * would require many years of health, of leisure, and of perseverance". To do profound investigation (in a field as complex as is the flow of water in open channels) in one's spare time presents a dilemma. Many an experimenter has wondered why the purse-strings of otherwise generous affluence are not loosened for practical engineering research.

It is with regret that the writer closes the discussion without giving fuller evidence of having carefully read all the valuable contributions. Probably the discussions and the wealth of hydraulic engineering literature in the numerous references furnish as nearly an up-to-date view of the subject as is practically possible.

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Paper No. 1712

ANALYSIS OF ARCH DAMS BY THE TRIAL LOAD METHOD

By C. H. Howell,* M. Am. Soc. C. E., and the Late A. C. Jaquith,† Esq.

With Discussion by Messrs. B. F. Jakobsen, George R. Rich, C. A. P. Turner, F. H. Constant, F. W. Hanna, L. J. Mensch, William Cain, Fred A. Noetzli, Fredrik Vogt, J. L. Savage and Ivan E. Houk, A. Floris, and C. H. Howell.

Synopsis

This paper presents a method of arch dam analysis that has been gradually developed in the Designing Office of the United States Bureau of Reclamation. It has also been used by the Salt River Valley Water Users Association, the Middle Rio Grande Conservancy District, and by one of the writers for a high arch dam in Mexico. It is based on the assumption that an arch dam is composed of systems of horizontal arches and vertical cantilevers, that each system carries a proportion of the total load, and that the deflections of the two systems are equal.

The method consists in applying a series of assumed proportional loads to each system and computing their deflections. When a proportionate loading has been found that gives equal deflections at all points for both systems, the stresses are computed and are considered to be those which will obtain in the dam.

The mathematical treatment of the problem has been gradually developed and improved. The final form is given in the Appendix. If applied to the earlier designs, the numerical results would be slightly different, but the stresses and the shape of the loads would not be greatly changed.

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It is now generally conceded that the cylinder formula does not give correct results when applied to arch dams. It makes no allowance for temperature changes, rib-shortening, or the effect of cantilever action. The theory is wrong and the resultant designs are uneconomical. The lack of economy increases very rapidly as the size of the dam is increased.

No dam, designed as a cylinder, has failed as far as is known to the writers. However, it is probable that some of them are stressed nearly to the safe limit. Existing dams designed as cylinders are of moderate height, and the maximum allowable stresses assumed in their designs are relatively low. These facts probably account for the lack of failures.

Many arch dams of great height are now under consideration. Consequently, the development of a complete theory of design is greatly needed. This paper is not presented as a complete solution of the problem, but as a step toward it.

A somewhat generally accepted method of investigating arch dams is to consider that the water load is carried by two systems of elements. One system is vertical and is considered as a series of cantilevers. The other system is horizontal and is considered as a series of arches. The two systems have equal deflections at all points and carry proportional parts of the load. It is probable that they are not exactly at right angles to each other, especially near the abutments. This complicates the problem. The amount of error caused by assuming the systems normal to each other is not known.

Several methods have been proposed for determining the distribution of the load between the arches and the cantilevers. These methods generally have been based on the relative rigidity of the arches and the cantilever at the crown. Each arch element is assumed to carry a uniform load such that, when the remainder of the unit water load is placed on the crown cantilever, the computed arch deflections at the crown will equal the computed crown cantilever deflections. It is believed that such a procedure is entirely inadequate.

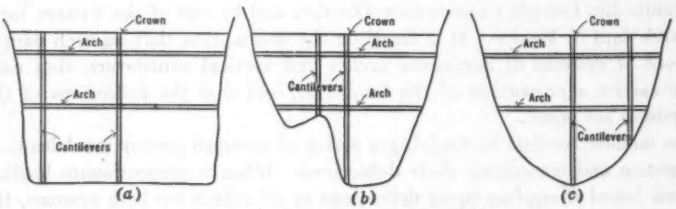

FIG. 1.—TYPICAL CANYON SECTIONS.

Consider a canyon section, as in Fig. 1 (*a*). The cantilevers at the crown and near the abutments, being of the same height and cross-section (assuming arch elements of uniform thickness), will theoretically deflect the same amounts under equal loads. Under a uniform load, the arch deflection at the crown will be much greater than that near the abutments. Hence, if the crown deflection of the arch equals the cantilever deflection at that point, it cannot be equal to that of the cantilevers at other points. Therefore, no uniform divi-

sion of load between the two systems can produce equal deflections at all points.

Consider a canyon section as in Fig. 1 (*b*). The cantilever at the hump, due to its greater stiffness, will clearly carry more of the load than the cantilever at the crown or maximum section. Therefore, the arch will carry less at the hump than at other points. In fact, the arch will undoubtedly be subject to a negative load at such a point. Some dams exist that have approximately this profile. Even in regular profiles similar to Fig. 1 (*c*) the arch loads were not found uniform in any of the detailed studies made by this method. All computations made thus far have indicated that uniform arch loads can obtain only under the most specialized conditions. It follows, therefore, that a general expression for the proportional loads on the two systems of elements must take into account the shape of the canyon. Such an expression has not yet been developed.

Computing the deflections and proportional loads at the crown alone, or at any other one section, therefore, means little in the study as a whole. The deflections and proportionate loads must be obtained at numerous points in each system in order to obtain even an approximate solution.

If the assumption that all the load is carried by the horizontal and vertical elements is accepted, there is certainly some definite division of the load that will give equal deflections for both systems at all points. This division can be found approximately by applying a series of trial proportionate loads to each system. If enough trials be made, equal deflections for arch and cantilever elements at all points will finally be found for any assumed temperature change. The computed stresses due to these final loads, when modified by Poisson's ratio, will probably be as close to the true stresses as those computed in gravity dams. It is believed they will be more nearly correct than stresses obtained by any method that ignores the effect of the cantilever elements or computes the cantilever effect at one section only.

This method has been in development for some time and is not yet considered as entirely complete. It is believed, however, that it has reached a stage where a description of it will be found interesting. A number of dams have been analyzed and designed by this system, and the results obtained for four of them are shown in the accompanying diagrams and tables. Unfortunately, no two of them are strictly comparable because of the gradual evolution and improvement of the mathematical treatment. When the plan was first evolved it was considered impracticable to do more than roughly approximate the actual conditions. For example, in the first analysis attempted, the stresses were computed on the assumption that reinforcing steel would have to be used to resist tension in the concrete and transmit tensile stresses across construction joints. As the methods of computation were improved during further studies it was found possible to go a step farther and eliminate computation of tension in the arch elements. This was done by varying the thickness of the arch and by assuming definite interior arches of regular shape at elevations where the first computations indicated tension. These interior arches are completely enclosed within the constructed arches and are entirely

in compression. The largest possible interior arch that could be drawn without showing tension, was used in the final analysis. Later, it was found possible to consider the arch action to be confined entirely to the irregular shape, whatever it might be, of the uncracked concrete that forms the actual interior arch. The load is assumed as applied to the constructed arch but entirely carried by the interior one. This method is now in use and the mathematics,

TABLE 1.—ARCH DATA, DAM NO. I.

Elevation in feet.	Thickness, in feet.	Angle.	Up-stream radius, in feet.	Down-stream radius, in feet.
4 830.0	5.00	141° 0′	197.08	192.08
4 845.4	5.00	141° 0′	197.08	192.08
4 815	7.84	108°12′	193.02	185.18
4 805	9.66	95°30′	189.15	179.49
4 795	11.48	83°00′	183.49	172.01
4 785	13.30	79°16′	177.10	163.80
4 775	15.12	77°30′	170.28	155.16
4 765	16.94	74°48′	162.80	145.86
4 755	18.75	72°08′	154.63	135.88

as given in the Appendix, has been adopted for the analysis of these irregularly shaped arches. The part of the base of the cantilever not in compression is likewise excluded in computing stability. No tensile stresses are now assumed in either cantilever or arches. This is believed to be essential for a correct analysis as it is practically impossible to secure an adequate anchorage for reinforcing steel in the arch abutments or in the bases of the cantilevers.

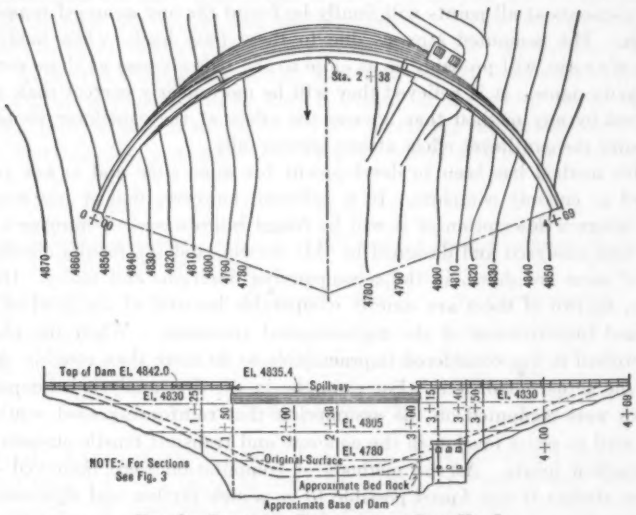

FIG. 2.—PLAN AND ELEVATION OF DAM NO. I.

Fig. 2 shows the layout and profile of Dam No. I, and Table 1 contains the arch data. This is a small dam designed and built in accordance with the cylinder formula for a stress of 25 tons per sq. ft. It is the first dam to be

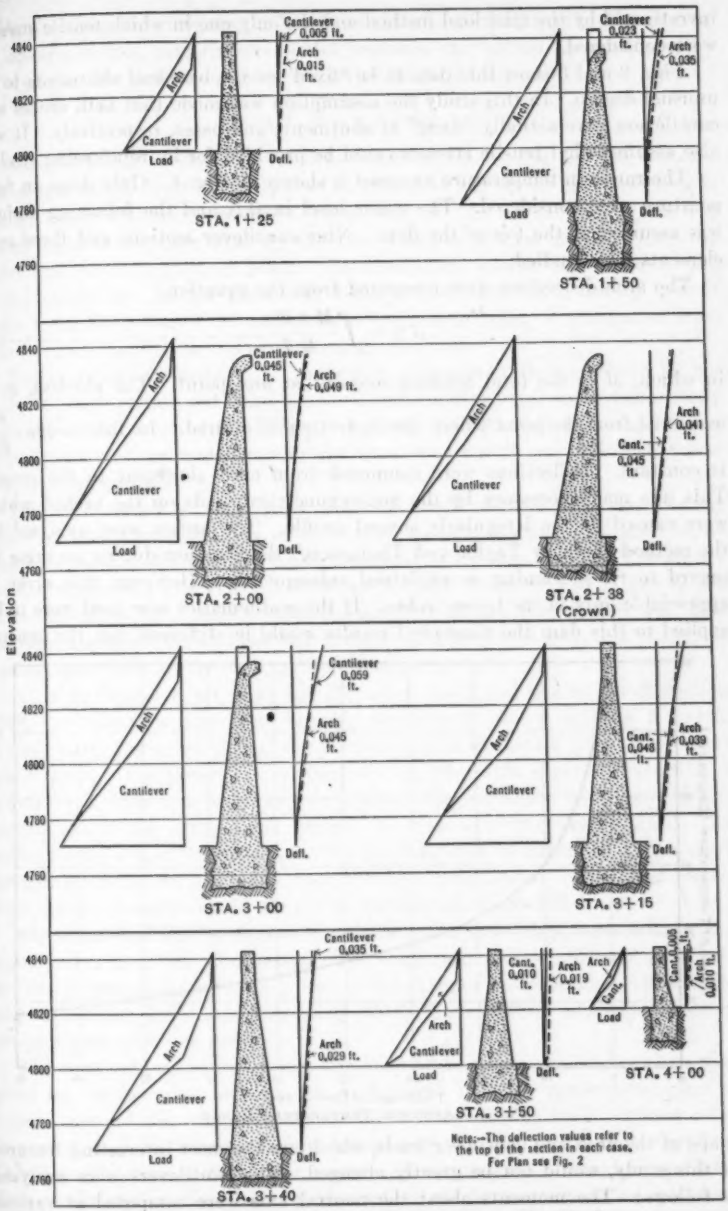

FIG. 3.—LOADS, SECTIONS, AND DEFLECTIONS OF DAM NO. I.

investigated by the trial load method and the only one in which tensile stresses were considered.

Figs. 2 and 3 show this dam to be "fixed" at the base and abutments to an unusual degree. In this study the assumption was made that both arches and cantilevers were actually "fixed" at abutments and bases, respectively. It was also assumed that tensile stresses could be provided for by reinforcing steel.

The range in temperature assumed is shown on Fig. 4. Only drops in temperature were considered. The water level in this and the following designs was assumed at the top of the dam. Nine cantilever sections and three arch elements were studied.

The arch deflections were computed from the equation,

$$\Delta y = \int \frac{M x \, d s}{E I} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

in which, M is the total bending moment at any point. The abscissa, x, is measured from the point where the deflection is desired. In this design, $\frac{d s}{E I}$ is constant. Deflections were computed from each abutment to the crown. This was made necessary by the non-symmetrical loads on the arches, which were caused by the irregularly shaped profile. The arches were analyzed by the method given by Taylor and Thompson,* thereby introducing an error in regard to rib-shortening as explained subsequently. However, this error is appreciable only in the lower arches. If the mathematics now used were to be applied to this dam the numerical results would be different, but the general

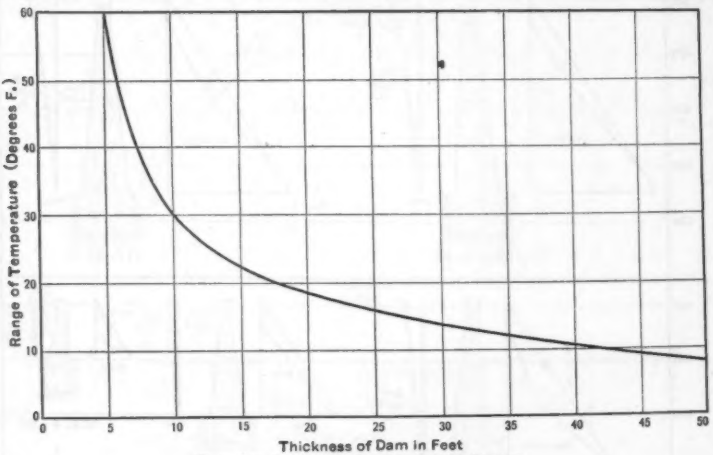

FIG. 4.—ASSUMED TEMPERATURE RANGE.

shape of the arch and cantilever loads, which are the most interesting features of this study, would not be greatly changed. The cantilevers were analyzed as follows: The moments about the neutral axis were computed at various

* In "Concrete Plain and Reinforced."

heights, loading the structure with the weight of concrete, weight of water on the up-stream face, and the horizontal pressure due to the assumed proportional load. The deflections were computed from the formula,

$$y = \int\int \frac{M}{E\,I}\, d\,x\, d\,x$$

Deflections due to shear are also included in studies for the latest dams. The computations for the cantilevers in this dam are somewhat more simple than in a dam without an overflow spillway notch. In the latter type, the top arches throw a negative load on the top of the cantilever because they are under little or no hydraulic head. The cantilevers, conversely, throw a load on the top arches. In this dam, with water at Elevation 4 842, the top arch is under a head of about 7 ft. and deflects correspondingly. No negative load, therefore, was applied to the cantilevers in this dam.

The modulus of elasticity, E, was taken as constant at 288 000 000 lb. per sq. ft. for both arches and cantilevers. There is, of course, much uncertainty as to the actual value of E. This uncertainty applies to all problems in concrete design and the usual assumption of a constant E was followed, therefore, in these studies.

Fig. 5 (*a*) and Fig. 5 (*b*) show two trial loads and the final load for the arches at Elevation 4 830. It is interesting to note the effect of the canyon profile on the shape of the arch loads, especially at Station 3 + 50. The abrupt change in the load line between Voussoir VI and Voussoir VII is due to the sudden break in the profile between Station 3 + 40 and Station 3 + 50. The diagrams also show the extreme sensitiveness of a long thin arch to a change of loading, especially if the loading is non-uniform.

After numerous trials, four for Elevation 4 805 and twelve for Elevation 4 830, the deflection lines were brought together as shown in Figs. 3 and 5. It was clearly indicated that if the process were continued a closer agreement would result. The poorest is at Station 3 + 50. This short stiff cantilever is only 10 ft. from one of maximum height. The deflection of the two under approximately the same loads cannot be even approximately equal. The arches cannot adjust themselves well to the difference in deflection in the short distance of 10 ft. At this point there will be a complicated condition of stress. Such abrupt changes should always be eliminated if possible.

The greatest difficulty found in adjusting the trial arch loads was in the upper arches near the abutments. The long thin arches are very sensitive, and slight changes of loads gave very great changes in deflections. Relatively large up-stream deflections frequently result from slightly excessive loads at the haunches, and *vice versa*.

The computed maximum stress was very high, very much higher than that given by the cylinder formula. It is believed that the constructed dam is stressed beyond the computed maximum, because it was built without reinforcement and, as tension is not taken care of, the arches have cracked. Therefore, they are resisting the compressive stresses on reduced dimensions. This condition is believed to be common to all dams designed as cylinders. On all the following designs it was found desirable to increase the arch thickness toward the abutments in order to obtain lower stresses.

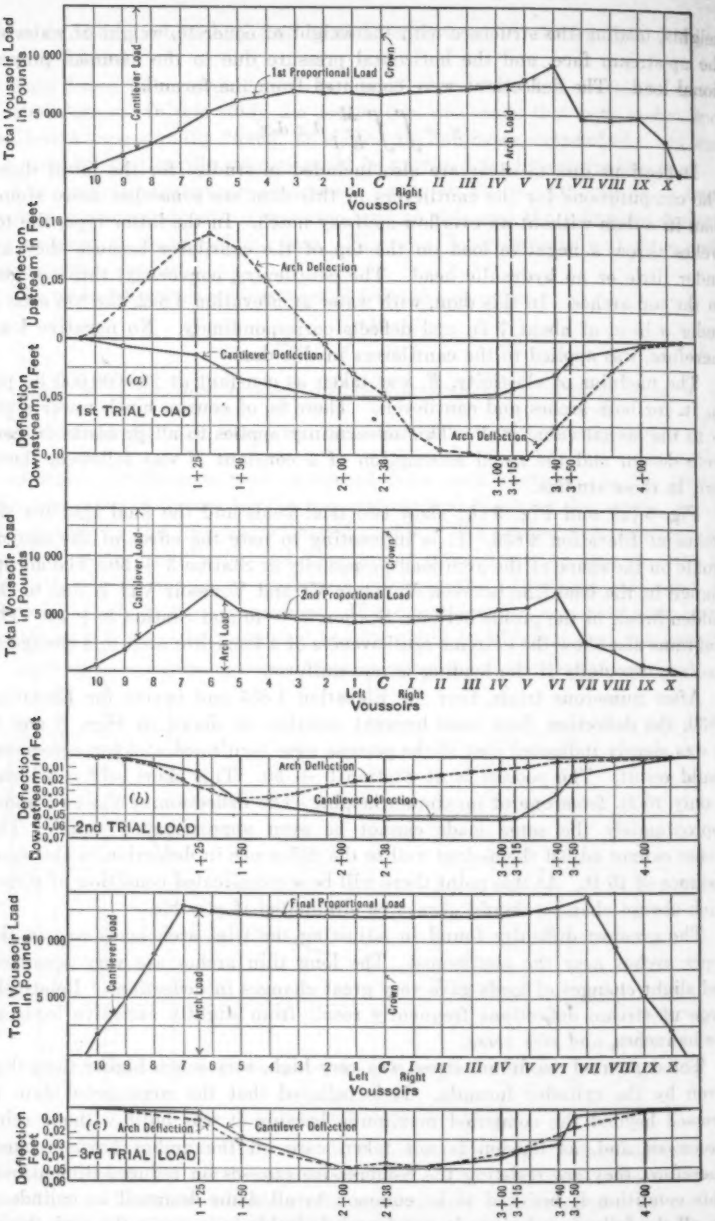

FIG. 5.—LOAD DEFLECTION DIAGRAM FOR DAM NO. I.

Dam No. II is a large arch dam, as yet (1927) in the preliminary design stage. It is interesting because of the very uniform V-shaped profile shown in Fig. 6. The arch data are arranged in Table 2. A dam designed by the cylinder formula was first proposed for this site. It was later analyzed by the trial load method, which showed that exceedingly high stresses would occur due to the faulty distribution of the concrete.

The design was then changed to that shown in Figs. 6 and 7. It was assumed that the profile would be made actually symmetrical by excavation and concrete footings, as shown in Fig. 6. This assumption reduced the work of analysis by one-half, as it was only necessary to compute the deflections of one-half the various arches and their respective cantilevers.

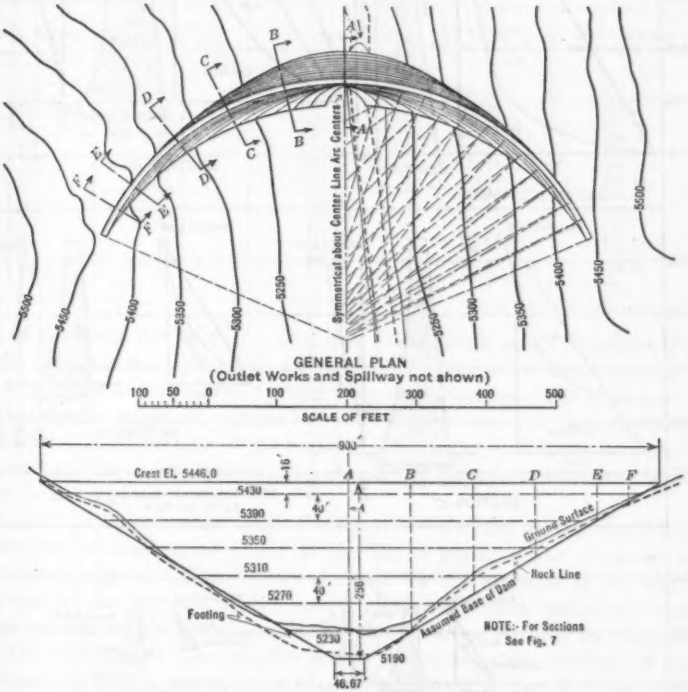

FIG. 6.—PLAN AND ELEVATION OF DAM NO. II.

The size of both systems of elements and the small depth of rock excavation precluded either system from being considered "fixed" in the sense that tension could be resisted by the grip of the rock on the concrete. Therefore, the arches were shaped so as to carry their loads entirely in compression. To accomplish this they were gradually thickened from crown to abutments, except at the top elevation. Both intrados and extrados are circular segments having different radii and centers. The cantilevers also were considered as incapable of resisting tension not only at their base but at any point. Unless reinforcing steel is used it is difficult to believe that tension could ever develop

in a dam due to the opening of the construction joints, even if it were considered that the concrete itself could resist tension. To simplify the calculations, the cantilevers for this and other dams were assumed to be 1 ft. thick

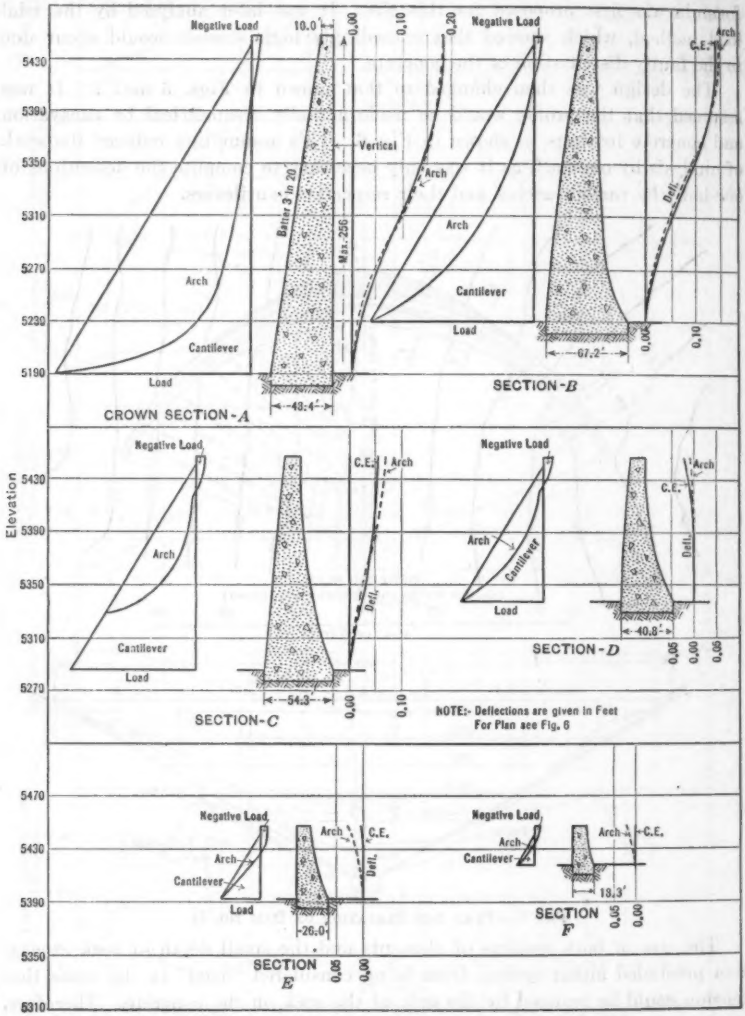

FIG. 7.—SECTIONS, LOADS, AND DEFLECTIONS, DAM NO. II.

throughout on lines radial to the crest of the dam. This is not strictly correct. The sections, except at the crown, are not exactly radial at the faces of the dam and the cantilevers are not of uniform thickness. The designs made thus far include whatever errors are due to the inaccurate assumptions mentioned.

The cantilever moments included the weight of concrete, water on the up-stream face, the negative load due to the unloaded arches at the top, and the assumed proportional part of the water load. These forces were taken as acting about the neutral axis of the sections at the various elevations considered. In cases where the resultant fell out of the middle third of the section, the moment of inertia was taken only for the portion of the section in compression, and the moments were considered as acting about the neutral axis of the compressed portion. The deflections were computed for each 20 ft. in height.

TABLE 2.—ARCH DATA, DAM NO. II.

Elevation, in feet.	Thickness of crown, in feet.	Angle.	Up-stream radius, in feet.	Down-stream radius, in feet.
5 446	10.0	132° 54′	388.00	378.00
5 430	12.4	126° 04′	386.45	369.28
5 410	15.4	117° 32′	384.11	337.39
5 390	18.4	109° 00′	381.20	343.55
5 370	21.4	100° 27′	377.87	326.61
5 350	24.4	91° 54′	372.84	308.91
5 330	27.4	83° 27′	366.84	285.15
5 310	30.4	75° 00′	358.14	256.70
5 290	33.4	65° 56′	351.69	228.00
5 270	36.4	56° 52′	337.17	196.70
5 250	39.4	48° 40′	316.99	143.11
5 230	42.4	40° 28′	284.73	92.77
5 210	45.4	32° 00′	231.65	53.33
5 190	48.4	.....	.....	.....

As in the study for Dam No. I, it was found much easier to adjust the loads for the lower elevations than for the higher. The top arches being only about 10 ft. thick and about 900 ft. long were very sensitive. The high cantilevers having the resultants outside the middle third were also very unstable. Moreover, the long lever arm of the negative load on the cantilevers (produced by the unloaded top arch) caused large changes in deflections with very small changes in the negative loads.

In the three lower arches (Elevations 5 310, 5 270, and 5 230) the thickness required to avoid an overhang at the higher portions was too great to avoid tension entirely. This tension develops on the extrados at the abutments and on the intrados at the crown. Interior arches, therefore, were assumed to form within the constructed arch and to act entirely in compression. The deflection of these interior arches should coincide with that of the cantilevers. It was in the analysis of the thick lower arches that the error in the usual method of computing rib-shortening was discovered. This error consists in computing the rib-shortening thrust from the average thrusts due to loads and temperature, without including the rib-shortening itself. If the rib-shortening is not included in obtaining the average, there will be negative thrusts in short thick arches. This error was corrected by a modification of the rib-shortening formula and the remaining arch analyses were similar to those used for Dam No. I. The computed results are believed to be quite close to those which would be obtained by the present mathematical treatment.

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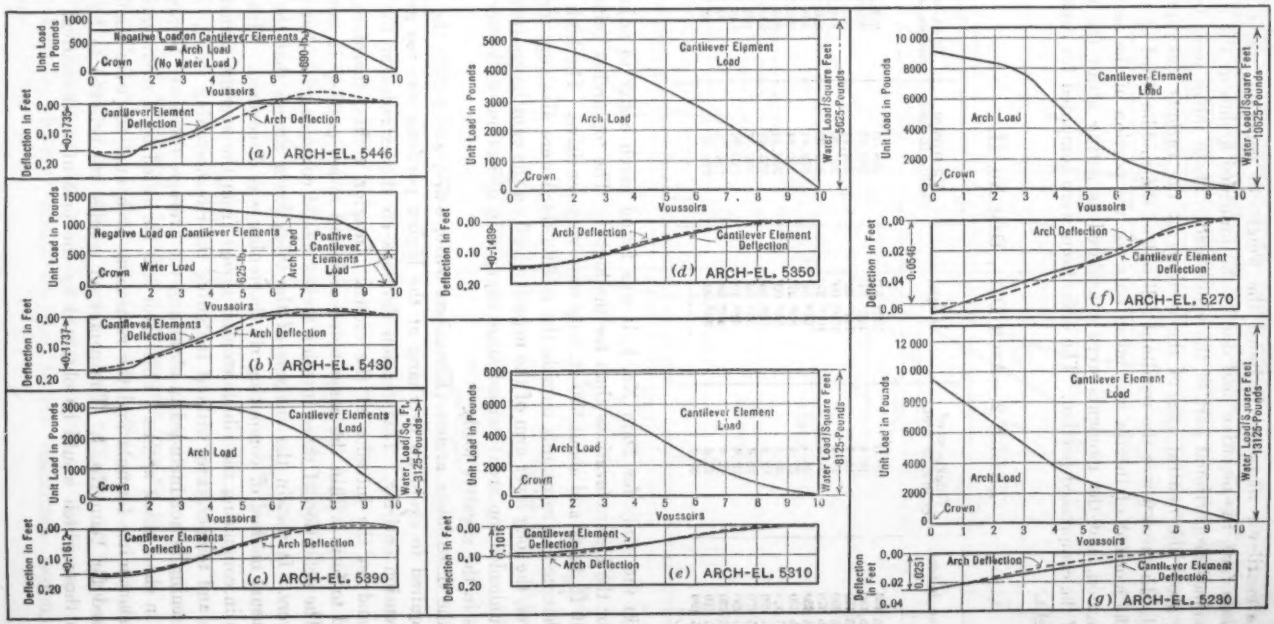

FIG. 8.—LOAD AND DEFLECTION CURVES, DAM NO. II.

Fig. 7 shows the vertical sections, the final proportionate water loads carried by each system, the negative load at the top, and the computed deflections. Fig. 8 (*a*) to Fig. 8 (*g*) shows the horizontal shape of the final arch loads and the deflections of the arches and cantilevers. Fig. 9 shows the developed elevation of the dam on which are drawn lines of equal deflections as computed for the arches. Fig. 10 shows the assumed interior arches for the three lowest elevations. They were found by removing from the original arches the parts in tension, and drawing symmetrical arches in the unarched portion. The reduced arches were then analyzed. This method is only approximate. It does not give definitely the largest interior arch that is free from tension.

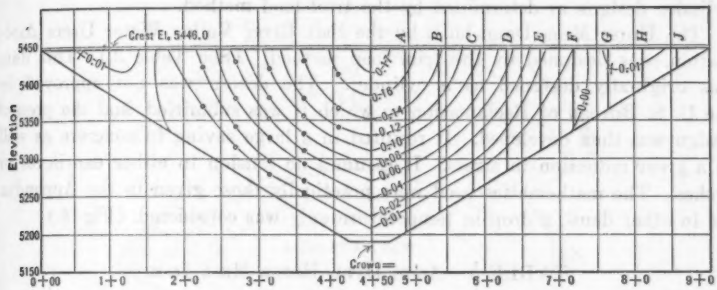

FIG. 9.—DEVELOPED ELEVATION OF DAM NO. II, SHOWING LINES OF EQUAL DEFLECTION (DEFLECTIONS, IN FEET).

At the time the studies of this dam were being made, it was considered too laborious to analyze the irregularly shaped interior arches. In the succeeding designs, however, this was done.

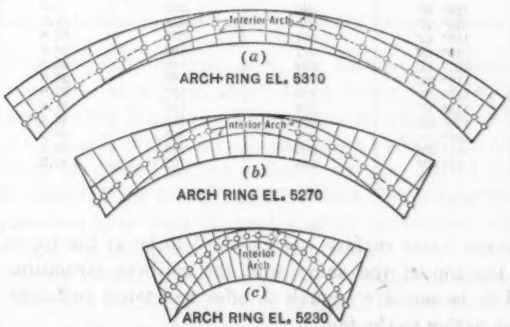

FIG. 10.—ASSUMED INTERIOR ARCHES, DAM NO. II.

The formation of the interior arches has been well stated by L. J. Mensch,* M. Am. Soc. C. E., William Cain,† M. Am. Soc. C. E., and also by B. F. Jakobsen,‡ M. Am. Soc. C. E.

* *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 308.
† *Loc. cit.*, p. 310.
‡ *Loc. cit.*, Vol. 90 (June, 1927), p. 475.

The greatest discrepancy between the cantilever and arch deflections is shown in Fig. 8 (*a*) and Fig. 8 (*b*) for Elevations 5 446 and 5 430. This is also shown in Fig. 7 for Cantilevers *D*, *E*, and *F*. These lines could have been brought more closely together by further trials. However, those already made, ten or more for each arch, showed relatively great variation in deflection and small changes in stress. Therefore, the further effort was not considered worth while.

As stated before, this design is a preliminary one only. It was made as an attempt to improve the original cylinder design. The saving in concrete amounts to 25%, and the stresses are less than one-half those in the cylinder designs as determined by the trial load method.

The Horse Mesa Dam, built by the Salt River Valley Water Users Association, was designed by the trial load method. (See Table 3.) This dam was originally designed as a cylinder. The design was not approved by the U. S. Bureau of Reclamation to which it was submitted, and the present design was then developed. It resulted in a large saving in concrete as well as a great reduction in stress. It assumes no tension in either cantilever or arches. The mathematics used were practically those given in the Appendix. As in other dams, a drop in temperature only was considered. (Fig. 4.)

TABLE 3.—ARCH DATA, HORSE MESA DAM.

Elevation of crest.	Central angle.	Up-stream radius, in feet.	Down-stream radius, in feet.	ARCH THICKNESS, IN FEET.	
				Crown.	Springing line.
1 921	112° 30′	240	232	8.0	8.0
1 897	114° 47′	235	218	11.6	14.4
1 875	116° 45′	231	206	14.8	20.1
1 850	119° 18′	227	198	18.6	26.8
1 825	121° 11′	224	181	22.3	33.4
1 800	118° 26′	220	168	26.0	40.9
1 775	117° 17′	216	157	29.0	46.0
1 750	116° 57′	213	146	31.8	51.5
1 725	118° 32′	210	136	34.8	56.5
1 700	106° 11′	207	123	36.5	61.0
1 675	91° 31′	202	113	38.5	59.8
1 650	76° 45′	195	102	40.2	57.3

The maximum water surface was assumed to be at the top of the parapet (5 ft. above the top of the dam), and the spillway structures at the ends were assumed to be massive enough to offer resistance sufficient for the dam to develop arch action to the top.

This design proved to be well proportioned, the average compression in the arches being 286 lb. per sq. in., and in the cantilevers, 222 lb. per sq. in. The difference in the maximum compression of the two elements was only 12 lb. The profile of this dam is irregular as was the case with Dam No. I, and again this irregularity is strikingly reflected by the shape of the arch loads.

On account of the thin section at the crown practically no load is taken by the cantilever. This condition exists at the abutments where, on account

of the almost perpendicular sides, the cantilevers cannot deflect; between these points the cantilevers carry a fair proportion at the bottom, but practically nothing at the top, of the dam.

In the analysis of this dam the effective section of the arches was considered to be only that portion in compression. Where tension developed, it was eliminated by assuming reduced sections until one was found that had practically a zero stress along the lines of the discarded areas. This condition occurred only in the lower portion of the dam where the arches are relatively thick. These reduced sections are shown in Fig. 11 and are not symmetrical, due to the unsymmetrical loading of the arches.

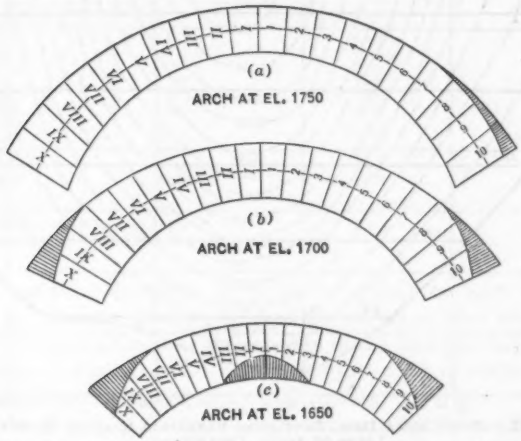

FIG. 11.—PLANS OF ARCHES AS ASSUMED TO ELIMINATE TENSION, HORSE MESA DAM.

Analysis discloses that, in this case, the maximum deflection does not occur at the crest, but at a point about 45 ft. below the crest and near the crown of the arch. (See Fig. 12.) This was to be expected as the only load on the arch at the top is that due to the negative load on the cantilevers. A similar condition of deflection was deduced by Mr. Alfred Stucky in his analysis of the Jogne Dam, Switzerland.* Since the designs described were made, the dimensions have been changed slightly to conform with conditions that have developed in the process of construction.

Table 4 gives the results of the analysis of the Gibson Dam on Sun River, Montana, which is now (1927) being built by the U. S. Bureau of Reclamation. This site has been under consideration for a long time, but hitherto only for a gravity dam. The shape of the profile does not clearly indicate the suitability of an arch dam. However, by taking advantage of the effect of the cantilever action in reducing water loads, as determined by the trial load method, the present design is responsible for a saving of more than 41 000 cu. yd. of concrete over the gravity design. In contrast to the three foregoing dams, the Gibson Creek Dam has a vertical up-stream face and a constant up-stream

* "Study of Arched Dams," *Bulletin technique de la Suisse Romande*, 1922.

radius. The previously considered designs are of the variable radius type. Because of the wide flat-bottomed profile (Fig. 13), a large proportion of the cantilevers are of approximately maximum height. In a variable radius design the crown cantilever has a vertical down-stream face and an inclined up-stream one. The adjacent sections approximate this shape. They are very inefficient in resisting thrusts on the up-stream face. The same area

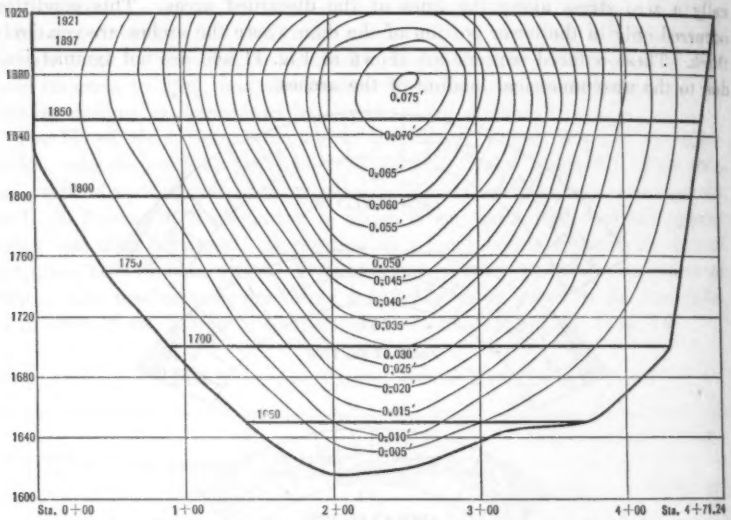

FIG. 12.—HORSE MESA DAM: DEVELOPED ELEVATION LOOKING UP STREAM, LINES OF EQUAL DEFLECTION.

with vertical up-stream and inclined down-stream faces is much stiffer. Therefore, if cantilever action is to be considered in profiles similar to that of the Gibson Dam, it will usually be found that the vertical up-stream face, or constant-radius type, is the most economical. This is not true, however, with narrow V-shaped profiles where the bottom arches are short and relatively more efficient, and the maximum height obtains for only a small proportion of the profile.

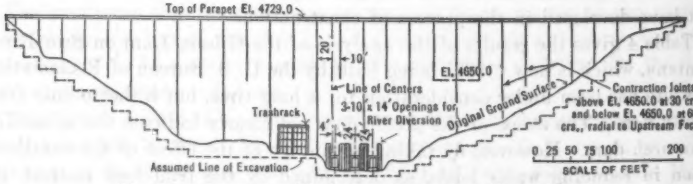

FIG. 13.—DEVELOPED UP-STREAM ELEVATION, GIBSON DAM.

The mathematics used are those given in the Appendix and represent the latest development. The maximum computed stress is 600 lb. per sq. in. and occurs at Elevation 4 550. The maximum arch stress is 300 lb. per sq. in.

and occurs at the crown at Elevations 4 675 and 4 650. Several arches at the lower elevation were analyzed with their full water load. These computations showed a maximum stress of approximately 1 000 lb. per sq. in. The computed stresses on this dam are believed to be the most accurately determined of all those studied thus far.

TABLE 4.—ARCH DATA, GIBSON DAM.

Station on line of centers.	Elevation, in feet.	Thickness on line of centers, in feet.	Down-stream radius, in feet.
0 + 00*	.....	.....	.....
0 + 00	4 725.5	15.0	390.00
0 + 00	4 717.5	16.57	388.43
0 + 07.5	4 700.0	20.0	377.5
0 + 18	4 675.0	25.0	362.0
0 + 34	4 650.0	30.0	341.0
0 + 48.8	4 625.0	36.2	320.0
0 + 66†	.....	.....	.....
0 + 74.5	4 600.0	44.5	286.0
0 + 96.9	4 575.0	55.1	253.0
1 + 17.0	4 550.0	71.5	216.5
1 + 17.0	4 530.0	87.0	201.0

* Center for up-stream radius of 405 ft.
† Station 5 + 34 base-line. Central angle equals 24° 00'.

In all the dams described, provision was made to leave openings for closing plugs. These plugs are poured in the coldest weather practicable, and as long a time as possible after the concrete in the dam has been placed so as to compensate, to some degree, for the shrinkage of the concrete. Other than this no consideration has been given to shrinkage.

In this design, due to the large relative importance of the cantilever action as compared to the arches, an arbitrary assumption for uplift was made. It was considered that the full head at the up-stream face diminished to zero or tail-water elevation at the down-stream face and acted over one-half the base of the section. In addition, a drainage system was provided.

No attempt was made in any of these studies to consider foundation deformation or swelling of the concrete due to water soakage. Both these actions undoubtedly modify deflections and stresses to some degree. The writers believe that an attempt to include foundation deformation would be unwarranted from a practical standpoint because of the indeterminate and, in many cases, the variable bearing power of the rock itself.

At present, so little is actually known about the changes in volume of concrete due to water soakage that the writers have not tried to include its effect in this method. If, however, one should desire to provide for the effect of water soakage there is no reason why the additional assumption could not be incorporated in the trial load method.

The action of gravity, however, is definitely known. The resistance offered by the cantilevers can be determined probably more accurately than that of the arches. The writers believe that action of cantilevers should not be neglected in designing arch dams. They certainly carry a part of the load and just as certainly modify the arch loads; and this fact should be recognized

in designing the arches. The trial load method offers a practical means of doing this.

Conclusions

In conclusion, the writers desire to repeat that this paper is not offered as a rigid and complete solution of the problem of arch dam design, but as a step toward it.

It is believed, however, that four conclusions are indicated:

(1) If the cantilevers act the arch elements are not under uniform loads.

(2) Computing proportional loads and deflections for each system at the crown, or at any other one point, means little or nothing in the solution of the problem as a whole.

(3) Rational formulas for a complete solution must include terms that express the shape of the canyon.

(4) An approximately accurate analysis can be made by the use of the method herein proposed.

The designs and studies of all the dams described were made under the direction of J. L. Savage, M. Am. Soc. C. E., Designing Engineer, U. S. Bureau of Reclamation. Julian Hinds, M. Am. Soc. C. E., as Assistant Designing Engineer, developed the final form of the mathematics and assisted throughout in perfecting this method. Ivan E. Houk and J. J. Doland, Members, Am. Soc. C. E., and Mr. R. E. Glover, Engineers, U. S. Bureau of Reclamation, assisted in the designs of the Gibson Dam.

APPENDIX

The following discussion involves the application of the elastic theory to arches both symmetrical and unsymmetrical in form. The underlying principles are the same as those given in various textbooks on concrete. The formulas deduced in textbooks, however, are generally subjected to simplifications that limit their usefulness in the study of arched dams. An attempt is here made to develop the formulas in a perfectly general way. The resulting equations are rather forbidding in appearance, but are, in fact, comparatively simple to apply.

Fig. 14 represents one side of an arch, the influence of the other side upon it being replaced by the introduction of proper forces or reactions at the crown. A load, W, placed on the arch will cause the crown point of the arch axis to be displaced. The displacement will be due partly to the bending of the arch ring, and partly to a change in the length of the center line of the arch.

In general, a load placed on the arch will produce a moment at every point along the ring. These moments cause the arch ring to bend.

The arch may be considered as made up of a series of short sections, or voussoirs. The bending of any short length, such as that having its center at D, will cause an angular displacement of the part ring, $A\ C$, and a displace-

ment, in space, of the crown point at C. The movement at C will be equal to $E\,C\,\Delta\,\theta$, plus the actual displacement of the point, E, $\Delta\,\theta$ being the change in angle between the two end faces of the voussoir. If the voussoirs are short, the movement is approximately $D\,C\,\Delta\,\theta$. From mechanics:

$$\Delta\,\theta = \frac{M\,s}{E\,I} \qquad (1)$$

in which, M is the bending moment, s is the length of the voussoir, E is the modulus of elasticity, and I is the moment of inertia of the arch ring. Equation (1) is strictly accurate only when M and I are constant throughout the voussoir, or when the length, s, is infinitesimal. However, it is assumed to serve with sufficient accuracy in practical cases if average values of M and I are taken for each voussoir. A further simplification is often made by assuming that M and I computed at the middle of the voussoir are equivalent to the average values for the voussoir.

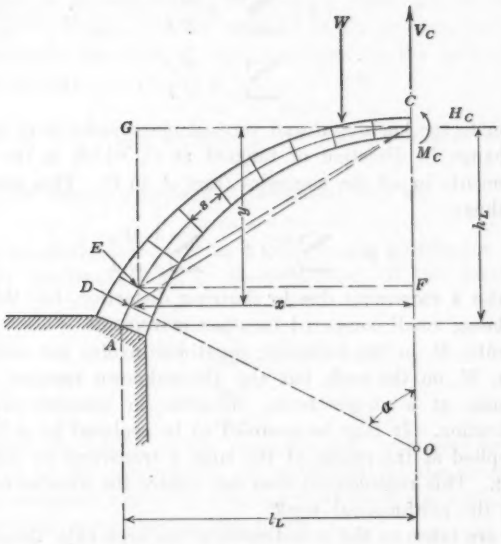

FIG. 14.

The angular movement, $\Delta\,\theta$, will be considered positive in a contra-clockwise direction, and M will be positive when the resulting tension is on the intrados side of the arch ring.

The path of motion at C will be determined by the direction of the line, $D\,C$. The use of angles is avoided by considering the vertical and horizontal motions separately. The vertical motion of the point, C, will be approximately equal to that of the point, F, or

$$\Delta\,y = x\,\Delta\,\theta = x\,\frac{M\,s}{E\,I} \qquad (2)$$

in which, Δy is the vertical movement of C, considered positive when upward; and x is the horizontal distance from C to D, considered positive regardless of direction of measurement.

The horizontal movement of C will be approximately equal to that of the point, G, or

$$\Delta x = -y\,\Delta\theta = -y\,\frac{M\,s}{E\,I} \quad \ldots\ldots(3)$$

in which, Δx is the horizontal movement of C, considered positive to the right, and y is the vertical distance from D to C, considered positive in all cases. The terms, horizontal and vertical, are used for convenience, although in arched dams the ordinate, y, is never vertical.

If Equations (2) and (3) are applied successively to each voussoir from A to C and the results added algebraically, the total movement of the point, C, due to bending, will be determined. The process may be expressed as follows:

$$\Delta y = \sum_{C}^{A} x\,\frac{M\,s}{E\,I} \quad \ldots\ldots(4)$$

$$\Delta x = -\sum_{C}^{A} y\,\frac{M\,s}{E\,I} \quad \ldots\ldots(5)$$

In addition to the horizontal and vertical movements there is an angular motion, or change in direction of tangent at C, which is the sum of the angular movements in all the voussoirs from A to C. This movement may be expressed thus:

$$\Delta\phi = \sum_{C}^{A} \Delta\theta = \sum_{C}^{A} \frac{M\,s}{E\,I} \quad \ldots\ldots(6)$$

There is also a movement due to shearing distortion, but this is usually neglected as being small compared to other movements.

The moments, M, in the foregoing equations involve not only the known external loads, W, on the arch, but also the unknown reaction at C. This reaction consists of a single force, unknown in amount, direction, and point of application. It may be assumed to be replaced by a horizontal or axial force applied at the center of the ring, a transverse or shearing force, and a moment. This replacement does not reduce the number of unknowns, but simplifies the arithmetical work.

If x and y are taken as the co-ordinates of the arch axis, then the moment of all forces about any given point between A and C (Fig. 14) will be:

$$M = M_C + H_C\,y + V_C\,x - M_L \quad \ldots\ldots(7)$$

in which, M_C, H_C, and V_C are, respectively, the moment, thrust, and shear at the crown, and M_L is the moment, about the point in question, of all external loads, W, between the crown and the point. All factors are considered positive when acting as shown in Fig. 14. The use of the minus sign before M_L in this case is not strictly mathematical, but conforms to usual practice. Actually, M_L is added algebraically, but, since it is in all usual cases a negative quantity, it is customarily computed without regard to algebraic sign and then subtracted.

Substituting this value for M in Equations (4), (5), and (6), yields three equations, as follows:

Change in y at crown,

$$\Delta y = \sum_C^A \frac{x\,s}{E\,I}(M_C + H_C\,y + V_C\,x - M_L) \quad \ldots\ldots (8)$$

Change in x at crown,

$$\Delta x = -\sum_C^A \frac{y\,s}{E\,I}(M_C + H_C\,y + V_C\,x - M_L) \quad \ldots\ldots (9)$$

Change in direction at crown,

$$\Delta \phi = \sum_C^A \frac{s}{E\,I}(M_C + H_C\,y + V_C\,x - M_L) \quad \ldots\ldots (10)$$

Equations (8), (9), and (10) give the movement at C produced by bending only, the length of the center line being assumed to be constant. If this length changes, as it always does in practice, there will be an additional movement. If the length, $A\ E$, is changed by an amount, Δs, other things remaining constant, the point, E, and, consequently, the point, C, will move vertically an amount:

$$\Delta y = \Delta s \sin \alpha \quad \ldots\ldots (11)$$

in which, α is the angle that a radius to the point, D, makes with the crown radius.

Likewise, the resulting horizontal movement will be,

$$\Delta x = \Delta s \cos \alpha \quad \ldots\ldots (12)$$

The total resulting movement at C will be the summation of the movements due to the lengthening or shortening of all the voussoirs between A and C, or,

$$\Delta y = \sum_C^A \Delta s \sin \alpha \quad \ldots\ldots (13)$$

$$\Delta x = \sum_C^A \Delta s \cos \alpha \quad \ldots\ldots (14)$$

A change in the length of the voussoirs may be caused by a change in temperature or by the compression (or extension) of the fibers by an axial thrust. The movement caused by a temperature change is readily determined. The total vertical movement at the crown is,

$$\Delta y = h_L\,c\,t \quad \ldots\ldots (15)$$

in which, h_L is the rise of the arch, c is the coefficient of temperature expansion, and t is the temperature change.

Likewise, if l_L is the length of the half span, the horizontal movement at the crown is,

$$\Delta x = l_L\,c\,t \quad \ldots\ldots (16)$$

The change in length in any voussoir due to thrust is:

$$\Delta s = -\frac{H s}{E\,T} \quad \ldots\ldots (17)$$

in which, H is the average axial thrust throughout the voussoir, and T is the average radial thickness of the arch ring, the thickness normal to the

plane of the paper in Fig. 14 being unity. The minus sign is required if both a compressive thrust and a lengthening of the fibers are considered positive. The value of H at any point is the combination of the components of H_C, V_C, and the external loads between the crown and the point, acting in the direction of the axis at the point. If H_L be chosen to represent the sum of the components of all external forces between the point and the crown, parallel to the axis at a given point, the thrust at that point is:

$$H = H_C \cos \alpha - V_C \sin \alpha + H_L \ldots\ldots\ldots\ldots\ldots\ldots (18)$$

Equation (17), therefore, may be written thus:

$$\Delta s = - \frac{s}{E\,T} (H_C \cos \alpha - V_C \sin \alpha + H_L) \ldots\ldots\ldots\ldots (19)$$

Inserting this value for Δs in Equations (13) and (14), the total movement of C due to elastic change in length of center line of arch may be expressed thus:

$$\Delta y = - \sum_C^A \frac{s \sin \alpha}{E\,T} (H_C \cos \alpha - V_C \sin \alpha + H_L) \ldots\ldots\ldots (20)$$

$$\Delta x = - \sum_C^A \frac{s \cos \alpha}{E\,T} (H_C \cos \alpha - V_C \sin \alpha + H_L) \ldots\ldots\ldots (21)$$

Adding Equations (8), (15), and (20), the total vertical movement at C is found to be:

$$\Delta y = \sum_C^A \frac{x\,s}{E\,I} (M_C\, H_C\, y + V_C\, x - M_L) + h_L\, c\, t$$

$$- \sum_C^A \frac{s \sin \alpha}{E\,T} (H_C \cos \alpha - V_C \sin \alpha + H_L) \ldots\ldots\ldots\ldots (22)$$

The total horizontal movement, found by adding Equations (9), (16), and (21), is:

$$\Delta x = - \sum_C^A \frac{y\,s}{E\,I} (M_C\, H_C\, y + V_C\, x - M_L) + l_L\, c\, t$$

$$- \sum_C^A \frac{s \cos \alpha}{E\,T} (H_C \cos \alpha - V_C \sin \alpha + H_L) \ldots\ldots\ldots\ldots (23)$$

Equations (10), (22), and (23) are independent, but cannot be solved since they contain the six unknowns, Δx, Δy, $\Delta \phi$, H_C, V_C, and M_C. However, the right-hand half of the arch may be treated in the same manner and three similar equations obtained involving the same six unknowns, thereby making it posssible to determine the reaction at the crown.

The equations for the right-hand part of the arch will be identical with those just developed for the left-hand part, except as to certain signs. A positive moment on the right will cause rotation in a clockwise, or negative, direction. For this reason, $\Delta \theta$ in Equation (1) and $\Delta \phi$ in Equations (6) and (10) become negative. Since distances and co-ordinates are considered positive regardless of direction, the sign of Δy will not change in any equation. A positive moment will produce a Δx-movement toward the right. Therefore, the negative sign will be dropped in Equations (3), (5), and (9). Since a positive shear is assumed to re-act upward on the left-hand part of the

arch, it must re-act downward on the right-hand half. It will produce, therefore, a negative bending moment, and the term, $V_C x$, in Equation (7) will become negative. Likewise, the effect on the thrust will be reversed and the V_C sin α-term in Equation (18) will become positive. The effect of Δs on Δx will be reversed, and a reversal of sign is required in Equations (12), (14), and (16). When these changes have been made and carried through the derived equations, a new set of equations applicable to the right side of the arch are obtained.

Equations (10), (22), and (23), on the left side, and the similar equations, (10), (22), and (23), on the right side, are independent, and contain only six unknowns. They are, therefore, capable of simultaneous solution. The crown movements, Δy, Δx, and $\Delta \phi$, are necessarily equal for the two sides of the arch. They may be eliminated at once by equating Equations (10) left, and (10) right, (22) left, and (22) right, and (23) left, and (23) right. The resulting equations are as follows:

$$\sum_C^A \frac{s}{E\,I}(M_C + H_C\,y + V_C\,x - M_L) = -\sum_C^B \frac{s}{E\,I}(M_C + H_C\,y - V_C\,x - M_R) \quad \ldots\ldots(24)$$

$$\sum_C^A \frac{x\,s}{E\,I}(M_C + H_C\,y + V_C\,x - M_L) + h_L\,c\,t - \sum_C^A \frac{s \sin\alpha}{E\,T}(H_C \cos\alpha - V_C \sin\alpha + H_L) = \sum_C^B \frac{x\,s}{E\,I}(M_C + H_C\,y - V_C\,x - M_R) + h_R\,c\,t - \sum_C^B \frac{s \sin\alpha}{E\,T}(H_C \cos\alpha + V_C \sin\alpha + H_R) \quad \ldots\ldots(25)$$

$$-\sum_C^A \frac{y\,s}{E\,I}\left(M_C + H_C\,y + V_C\,x - M_L\right) + l_L\,c\,t - \sum_C^A \frac{s \cos\alpha}{E\,T}\left(H_C \cos\alpha - V_C \sin\alpha + H_L\right) = \sum_C^B \frac{y\,s}{E\,I}\left(M_C + H_C\,y - V_C\,x - M_R\right) - l_R\,c\,t + \sum_C^B \frac{s \cos\alpha}{E\,T}\left(H_C \cos\alpha + V_C \sin\alpha + H_R\right) \quad \ldots\ldots(26)$$

If $\sum_C^A$ and $\sum_C^B$ are designated, respectively, by $\sum_L$ and $\sum_R$, Equations (24), (25), and (26) simplify as follows:

$$M_C\left(\sum_L \frac{s}{I} + \sum_R \frac{s}{I}\right) + H_C\left(\sum_L \frac{s\,y}{I} + \sum_R \frac{s\,y}{I}\right) + V_C\left(\sum_L \frac{s\,x}{I} - \sum_R \frac{s\,x}{I}\right) = \sum_L \frac{M_L\,s}{I} + \sum_R \frac{M_R\,s}{I} \quad \ldots\ldots(27)$$

$$M_C\left(\sum_L \frac{x\,s}{I} - \sum_R \frac{x\,s}{I}\right) + H_C\left(\sum_L \frac{x\,y\,s}{I} - \sum_L \frac{s \sin\alpha \cos\alpha}{T} - \sum_R \frac{x\,y\,s}{I} + \sum_R \frac{s \sin\alpha \cos\alpha}{T}\right) + V_C\left(\sum_L \frac{x^2\,s}{I} + \sum_R \frac{x^2\,s}{I} + \sum_L \frac{s \sin^2\alpha}{T} + \sum_R \frac{s \sin^2\alpha}{T}\right) = \sum_L \frac{M_L\,x\,s}{I} - \sum_R \frac{M_R\,x\,s}{I} + \sum_L \frac{H_L\,s \sin\alpha}{T} - \sum_R \frac{H_R\,s \sin\alpha}{T} - c\,t\,E\,(h_L - h_R) \quad \ldots\ldots(28)$$

$$M_C\left(\sum_L \frac{y\,s}{I} + \sum_R \frac{y\,s}{I}\right) + H_C\left(\sum_L \frac{y^2\,s}{I} + \sum_R \frac{y^2\,s}{I} + \sum_L \frac{s \cos^2\alpha}{T} + \sum_R \frac{s \cos^2\alpha}{T}\right) + V_C\left(\sum_L \frac{x\,y\,s}{I} - \sum_R \frac{x\,y\,s}{I} - \sum_L \frac{s \sin\alpha \cos\alpha}{T} + \sum_R \frac{s \sin\alpha \cos\alpha}{T}\right) = \sum_L \frac{M_L\,y\,s}{I} + \sum_R \frac{M_R\,y\,s}{I} - \sum_L \frac{H_L\,s \cos\alpha}{T} - \sum_R \frac{H_R\,s \cos\alpha}{T} + c\,t\,E\,(l_L + l_R) \quad \ldots\ldots(29)$$

After the constant terms and the constant coefficients of M_C, H_C, and V_C are found, Equations (27), (28), and (29) are readily solved. If desired, the solution may be made in the general form, and expressions obtained from which the unknowns can be calculated directly, but the work is not expedited by this procedure.

After M_C, H_C, and V_C are found, the moments, thrusts, and shears at other points may be computed from Equations (7) left, (7) right, (18) left, and (18) right.

The horizontal and vertical components of the crown deflection are given by Equations (22) left, and (23) left, or by Equations (22) right, and (23) right. These formulas can be used for computing the deflection at points other than the crown, by carrying the summations from the abutment up only to the point at which the deflection is desired. The modulus of elasticity, E, is assumed as a constant, and may be placed outside the summation sign wherever convenient.

The deflections at intermediate points can be determined with less labor, however, by computing the movement within each voussoir separately, and taking the summation from the abutment toward the crown. The angular movement of any division, such as the one having its center at F, Fig. 14, is found by the summation of Equation (1), from A to F, thus:

$$\Delta\,\theta_F = \sum_F^A \frac{M\,s}{E\,I} \quad \ldots\ldots(30)$$

The actual linear movement of the upper end of the voussoir, due to this angular movement, is:

$$\Delta\,d_F = s \sum_F^A \frac{M\,s}{E\,I} \quad \ldots\ldots(31)$$

The movement, $\Delta\,d_F$, in Equation (31) is, practically, at right angles to the chord of the voussoir; that is, parallel to the radius through the center

of the voussoir. These movements for the various voussoirs are not in one direction, and cannot be added algebraically, as they stand. They may be combined graphically, or resolved into horizontal and vertical components, and then added. The components are obtained by multiplying Equation (31) by the cosine and sine of the angle to the center of the voussoir. Performing these multiplications and taking the summation from the abutment up, the total component movements of the upper end of the voussoir, F, are found to be:

$$\Delta x_F = \sum_F^A \left(s \cos \alpha \sum_F^A \frac{M s}{E I} \right) \quad \ldots\ldots\ldots\ldots\ldots\ldots (32)$$

$$\Delta y_F = \sum_F^A \left(s \sin \alpha \sum_F^A \frac{M s}{E I} \right) \quad \ldots\ldots\ldots\ldots\ldots\ldots (33)$$

Equations (30) to (33), inclusive, refer to movements due to bending only. The movements caused by change in length of the center line, due to thrust and temperature change, are parallel to the various chords. They are, therefore, normal to the movements due to bending and must be resolved separately. After the individual components for both bending and shortening have been found, they may be added separately, and then combined, or combined and then added. Where the work is done on a computing machine it is advantageous to determine the summation separately. The total movement in any voussoir from temperature and thrust, is,

$$\Delta s = s c t - \frac{H s}{E T} \quad \ldots\ldots\ldots\ldots\ldots\ldots (34)$$

Taking components and summating, the total horizontal and vertical movements at the upper end of the voussoir, F, due to temperature and rib-shortening, are

$$\Delta x_F = \sum_F^A \cos \alpha \left(s c t - \frac{H s}{E T} \right) \quad \ldots\ldots\ldots\ldots\ldots\ldots (35)$$

$$\Delta y_F = \sum_F^A \sin \alpha \left(s c t - \frac{H s}{E T} \right) \quad \ldots\ldots\ldots\ldots\ldots\ldots (36)$$

Equations (32) and (35) and Equations (33) and (36) may be combined if desired.

All the formulas deduced have been left in the general form. If the factor, s, is made constant for the entire arch, or for each side, it may be removed from the summation, or both s and I may be so removed if $\frac{s}{I}$ is constant. The modulus of elasticity, E, is presumably a constant, and may be placed outside the summation sign wherever convenient.

These equations do not differ essentially from those ordinarily given for the elastic arch, except in regard to the allowance for the elastic shortening of the arch rib. In the analysis of concrete bridge arches it is customary to base the rib-shortening stresses on the average thrust exclusive of the rib-shortening thrust. In long flexible arches this plan is sufficiently accurate for all practicable purposes. With short stiff arches, such as those encountered at the bottom of an arched dam, the magnitude of the rib-shortening thrust

becomes appreciable, and failure to make proper allowance in computing the average thrust is likely to lead to serious error. This approximation is eliminated from the equations herein deduced.

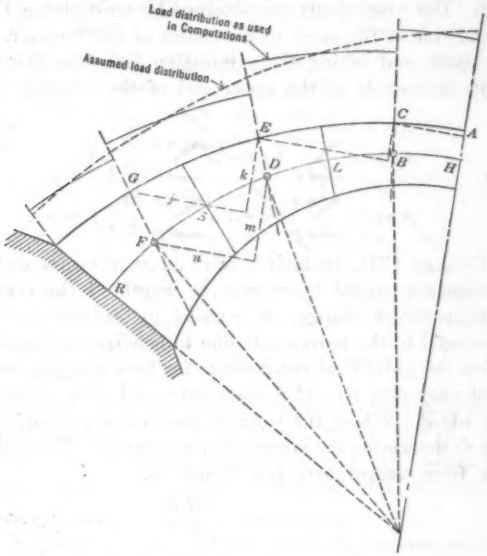

FIG. 15.

Applying this rule to Fig. 15, and considering the horizontal and vertical forces separately, the moment at F is, evidently,

$$M_F = M_D + n \sum_D^H p\ X + m \sum_D^H p\ Y + \frac{1}{2}\ n\ p\ j + \frac{1}{2}\ n\ p\ k \ldots (37)$$

In Equation (37), M_D is the total moment at D. $\sum_D^H p\ X$ is the vertical shear at D, $\sum_P^H p\ Y$ is the horizontal shear at D, X is the vertical load increment between D and F, and $p\ Y$ is the corresponding horizontal load increment. In Table 5, j and k are represented by X and Y, respectively, and n and m by Δx and Δy.

Accordingly, Equation (37) may be revised as follows:

$$M_F = M_D + \Delta x \left(\frac{1}{2}\ p\ X + \sum_D^H p\ X\right) + \Delta y \left(\frac{1}{2}\ p\ Y + \sum_D^H p\ Y\right). \quad (38)$$

Evidently, $\frac{1}{2}\ p\ X + \sum_D^H p\ X$ and $\frac{1}{2}\ p\ Y + \sum_D^H p\ Y$ are, respectively, the vertical and horizontal shear at S, and the moment equation may be further revised to read,

$$M_F = M_D + \Delta x \sum_S^H p\ X + \Delta y \sum_S^H p\ Y \ldots\ldots\ldots (39)$$

To facilitate the application of these formulas a computation form has been devised, as shown in Table 5. The arch used as an example is at Elevation 4 625 of the Gibson Dam, on the Sun River Project. (See, also, Table 4.)

In Table 5, Columns (1) and (4) represent basic data, taken from the drawing of the arch to be analyzed, T being the average thickness of the voussoirs, and α the radial angle from the crown to the center of the various voussoirs. Column (2) is used only in the derivation of the reciprocal of the moment of inertia, Column (3).

Column (5) is used for determining values of x and the vertical components of forces and movements. Column (6) is used for the computation of horizontal components of forces and deflections. Column (7) is for use in the computation of Column (12). Columns (8) and (9) are for use in the production of other columns containing these functions, and for insertion in the equations for determining the crown forces. The co-ordinates, x and y, Columns (10) and (12), are, respectively, the horizontal and vertical distances from the crown to the center of the voussoirs, as indicated in Fig. 14.

For a simple circular center line, x is equal to radius times sine α and y is equal to the radius times vers. α. The values of Δx and Δy, Columns (11) and (13), Table 5, may be obtained by taking the differences of consecutive values in Columns (10) and (12), respectively. For a non-circular, or compound arch center line, the values of the co-ordinates and their differences cannot be computed by the method outlined, and suitable changes in column designations and in the method of procedure will be required. Columns (14) and (15) are used in the preparation of other columns, and their sums are used in the determination of the crown forces.

The load co-ordinates, X and Y, Columns (16) and (17), Table 5, correspond to the differential co-ordinates, Δx and Δy, Columns (11) and (13), except that they are measured, or computed to the up-stream face of the dam. Where the water face and the center line of the arch are not concentric, the computations of these co-ordinates are complicated. They usually may be scaled with sufficient accuracy. The method of laying these co-ordinates out is shown on Fig. 15. The points, B, D, and F, are at the mid-points of the center lines of their respective voussoirs, and the lines, BC, DE, and FG, are radial to the water face of the dam. Obviously, the water load coming on the chord, FD, is included between the points, G and E, and similarly for the other chord lengths.

In Table 5, Column (18) contains one-half the average unit water loads for Divisions AC, CE, etc. The loads may be measured at the two ends of the divisions and averaged; or the measured, or computed, load at the center of each division may be taken as the average. Columns (19) and (20) show, respectively, one-half the vertical and horizontal loads on the voussoirs. Summations of the load components, from the crown toward the abutment, shown in Columns (21) and (22), are used to compute the water-load moments, M_L, and the water load thrusts, H_L, shown in Columns (23) and (24). A description of the method of computing these moments and thrusts will bring out the reasons for using half loads in Columns (18), (19), and (20).

TABLE 5.—COMPUTATIONS FOR LEFT HALF OF ARCH, GIBSON DAM.*

Point.	T.	T^2.	$\frac{1}{I}$.	α.	sin α.	cos α.	vers α.	$\frac{\cos\alpha}{T}$.	$\frac{\cos^2\alpha}{T}$.*	x.	Δx.	y.	Δy.	$\frac{Y}{I}$.	$\frac{y^2}{I}$.†	Load Co-ordinate.	
																X.	Y.
Column No..	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)
Crown.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
1	36.1	47 046	0.00025507	2° 19′	0.04042	0.99918	0.00082	0.027678	.....	14.58	14.58	0.30	0.30	0.0000765210	.....	15.2	0.4
2	36.3	47 832	0.00025088	6° 57′	0.12100	0.99265	0.00735	0.027346	.....	43.66	29.08	2.65	2.35	0.0006648320	.....	30.5	2.3
3	37.0	50 653	0.00023691	11° 35′	0.20079	0.97963	0.02037	0.026476	.....	72.44	28.78	7.35	4.70	0.0017412885	.....	30.3	4.7
4	38.0	54 872	0.00021869	16° 18′	0.27927	0.96021	0.03979	0.025269	.....	100.76	28.32	14.36	7.01	0.0031403884	.....	29.7	6.8
5	39.2	60 236	0.00019922	20° 51′	0.35592	0.93451	0.06548	0.023840	.....	128.42	27.66	23.62	9.26	0.0047055764	.....	29.2	9.2
6	40.8	67 917	0.00017669	25° 29′	0.43025	0.90271	0.09729	0.022125	.....	155.23	26.81	35.10	11.48	0.0062018190	.....	28.5	11.3
7	42.5	76 765	0.00015632	30° 07′	0.50176	0.86500	0.13499	0.020353	.....	181.03	25.80	48.70	13.60	0.0076127840	.....	27.6	13.5
8	44.7	89 314	0.00013436	34° 48′	0.57000	0.82165	0.17835	0.018381	.....	205.65	24.62	64.85	15.65	0.0086460660	.....	26.6	15.7
9	47.0	103 823	0.00011558	39° 23′	0.63450	0.77292	0.22708	0.016445	.....	228.93	23.28	81.93	17.58	0.0094694694	.....	25.5	17.7
10	49.6	122 024	0.00009834	44° 01′	0.69487	0.71914	0.28086	0.014499	.....	250.71	21.78	101.33	19.40	0.0099647922	.....	24.2	19.7
Springing line.....	.....	.....	.....	46° 20′	0.72337	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
Sigma.....	.....	.....	0.00184206	.....	.....	.....	.....	.....	0.208097298	.....	.....	.....	.....	0.0522265869	3.101191720	.....	.....

* Right half similar.

† Computed and added by machine.

TABLE 5.—(*Continued.*)

Point.	One-half unit water load = ½ *p*	Vertical water load = ½ *p X*.	Horizontal water load = ½ *p Y*.	Σ *p X*.	Σ *p Y*.	$\Sigma(\Delta x \Sigma p X + \Delta y \Sigma p Y) = M_L$.	$\frac{\sin\alpha \Sigma p X}{-\cos\alpha \Sigma p Y} = H_L$.	$\frac{M_L}{I}$.	$\frac{M_L y}{I}$.	$\frac{H_L \cos\alpha}{T}$.	$H_C y$.	$M = M_C + H_C y + V_C x - M_L$.	$\frac{H_C}{\cos\alpha}$.	$H = H_L + H_C \cos\alpha - V_C \sin\alpha$.	$\frac{1}{2}\frac{M}{I}$.
Column No..	(18)	(19)	(20)	(21)	(22)	(23)	(24)	(25)	(26)	(27)	(28)	(29)	(30)	(31)	(32)
Crown.....	.....	.....	.....	18 620	490	.....	.....	.....	.....	.....	.....	+5 379 826	.....	604 086	.....
1	1 225	18 620	490	37 240	980	271 627	526	.....	.....	.....	181 226	+5 289 425	603 591	604 117	+674.587
2	1 175	35 838	2 708	73 078	3 683	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
				108 916	6 386	2 405 390	6 840	.....	.....	.....	1 600 828	+4 575 264	599 646	606 486	+573.921
3	1 100	33 330	5 170	142 246	11 556	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
				175 576	16 726	6 553 543	18 869	.....	.....	.....	4 440 032	+3 266 315	591 781	610 650	+386.911
4	1 060	31 482	7 208	207 058	23 934	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
				238 540	31 142	12 585 208	36 714	.....	.....	.....	8 674 675	+1 469 298	580 049	616 763	+160.660
5	925	27 010	8 510	265 550	39 652	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
				292 560	48 162	20 297 498	59 120	.....	.....	.....	14 268 511	— 649 156	564 524	623 645	— 64.662
6	725	20 663	8 193	313 223	56 355	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
				333 886	64 548	29 841 957	85 386	.....	.....	.....	21 203 419	−2 758 712	545 314	630 700	−243.718
7	625	17 250	8 438	351 136	72 988	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
				368 386	81 424	39 393 876	114 410	.....	.....	.....	29 418 988	−4 595 062	522 534	636 944	−359.150
8	550	14 630	8 635	383 016	90 059	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
				397 646	98 694	50 233 153	145 566	.....	.....	.....	38 872 934	−5 980 393	496 847	641 913	−401.768
9	475	12 113	8 408	409 759	107 102	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
				421 872	115 510	61 655 196	178 398	.....	.....	.....	49 492 766	−6 782 604	466 910	645 306	−391.967
10	350	8 470	6 895	430 280	122 405	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
				438 688	129 300	73 401 351	211 846	.....	.....	.....	61 212 034	−6 809 491	434 422	646 268	−334.823
Springing line.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
Sigma.....	.....	.....	.....	.....	.....	.....	.....	41 457.477	2 379 526.85	15 937.04	....				

* Computed and added by machine.

TABLE 5.—(*Continued.*)

Point.	$\Sigma \frac{M}{I}$.	$\frac{s^2}{E} \Sigma \frac{M}{I}$.	Σ Column (34) $\cos \alpha$.	$-\frac{s}{E} \frac{H}{T}$	$\frac{s}{E} \frac{H}{T} - c\,t\,s$.	Σ — Column (37) $\sin \alpha$.	$y\,d =$ Column (35) + Column (38).	Σ — Column (34) $\sin \alpha$.	Σ — Column (37) $\cos \alpha$.	$x\,d =$ Column (40) + Column (41).	Radial deflection = Column (39) $\cos \alpha$ — Column (42) $\sin \alpha$.	Eccentricity = $\frac{M}{H}$ Column (29) $\frac{}{H}$ Column (31).	Column (31) $\frac{}{144\,T}$.	Column (32) $\frac{T}{144}$.	Stress, in Pounds per Square Inch.	
															Extrados, Column (45) + Column (46).	Intrados, Column (45) — Column (46).
Column No..	(33)	(34)	(35)	(36)	(37)	(38)	(39)	(40)	(41)	(42)	(43)	(44)	(45)	(46)		
Crown.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	+116	+162	+278	— 46
1	— 674.595	—0.001994	0.058413	0.001627	0.002677	0.009454	0.067867	0.022803	—0.022793	—0.00001	—0.068	+ 8.91	.....			
2	—1 923.103	—0.005684	0.056420	0.001693	0.002673	0.009346	0.065766	0.022722	—0.020118	—0.002604	—0.065	+ 8.76	+116	+145	+261	— 29
3	—2 883.935	—0.008524	0.050778	0.001650	0.002630	0.009023	0.059801	0.022034	—0.017465	—0.004560	—0.058	+ 7.54	.....			
4	—3 431.506	—0.010143	0.042428	0.001645	0.002625	0.008495	0.050925	0.020923	—0.014888	—0.005435	—0.047	+ 5.35	+113	+ 42	+155	+ 71
5	—3 527.501	—0.010427	0.032689	0.001612	0.002592	0.007761	0.040450	0.017490	—0.012368	—0.005122	—0.036	+ 2.38	.....			
6	—3 219.124	—0.009515	0.022945	0.001565	0.002545	0.006839	0.029784	0.013779	—0.009946	—0.003833	—0.025	— 1.05	.....			
7	—2 616.256	—0.007733	0.014332	0.001518	0.002498	0.005744	0.020076	0.009685	—0.007648	—0.002037	-0.016	— 4.37	+107	— 69	+ 38	+176
8	—1 855.343	—0.005484	0.007643	0.001455	0.002435	0.004491	0.012134	0.005805	—0.005487	—0.000318	—0.010	— 7.21	.....			
9	—1 061.613	—0.003138	0.003137	0.001891	0.002371	0.003103	0.006240	0.002679	—0.003487	+0.000808	—0.005	— 9.32	+100	—124	— 24	+224
10	— 334.823	—0.000990	0.000712	0.001320	0.002300	0.001598	0.002310	0.000688	—0.001654	+0.000966	—0.002	—10.51	.....			
Springing line.....							.....			.....	.....	.....	+ 90	—115	— 25	+205
Sigma.																

In order that the moments at the various points of diversion may be computed progressively, use is made of the rule that the moment at any point is equal to the moment at a preceding point, plus the shear at the preceding point, times the distance between the two points, plus the moment of any intervening loads.

If only the moments were desired the load co-ordinates might be measured, therefore, between the ends, rather than the centers of the voussoirs, and the full water loads used. However, the water thrusts are functions of the shears at the centers of the voussoirs. For this reason the water loads are calculated and summed by halves, giving the load summations at both the centers and the ends of the voussoirs. Additional columns might be provided between Columns (22) and (23), in which to record and combine $\Delta x \Sigma p X$ and $\Delta y \Sigma p Y$ before summing. Table 5, as shown, is designed for use in connection with a computing machine, and no advantage results from recording intermediate steps. The only disadvantage in this plan is that it causes some confusion as to where the computed sums shall be recorded. It will be noted that two horizontal lines are provided in Table 5 for each voussoir. Functions that apply at the centers of the voussoirs are recorded opposite the numerals 1, 2, 3, etc., in the "point" column. These numerals represent the numbers of the voussoirs. Functions pertaining to the joints between voussoirs are recorded in the intermediate horizontal lines. The increment in moment to any voussoir center is obtained by multiplying the shear at the preceding voussoir end by the preceding value of Δx or Δy, and, if recorded, would be set down in the horizontal line intermediate between the two points to which it applies. However, when summated, these differences become the moments at the points, and, for clearness, should be dropped a half space when recorded in Column (23). The first moment recorded in Column (23), which comes at the center of the first voussoir, is obtained as follows:

$$18\,620 \times 14.58 + 490 \times 0.30 = 271\,627$$

The second moment, which is at the center of the second voussoir, is obtained by adding

$$73\,078 \times 29.08 + 3\,683 \times 2.35$$

to the previous moment; and the remainder are computed in a similar manner to the center of the last voussoir. Column (23), therefore, may be produced by multiplying, successively, in horizontal pairs, the voussoir joint values in Columns (21) and (22) by the corresponding values in Columns (11) and (13), recording a half space down, in Column (13), the sum of the products after the completion of each pair of multiplications. The computing machine must not be cleared between multiplications.

The thrusts, H_L, in Column (24) are obtained from the figures in Columns (21) and (22), as indicated, without continuous summation, the values being recorded opposite the numbers from which they are derived.

The derivation of Columns (25) to (27), inclusive, is evident. The summations of these columns complete the data required for the determination of the crown forces, from Equations (27), (28), and (29). A convenient form for use in the computation of the constants in these equations is arranged at

the end of Table 3. Space is also provided in which the computations for M_C, H_C, and V_C may be made.

Having obtained the crown forces, the moments and thrusts at intermediate points are readily computed, as shown in Columns (28) to (31), inclusive.

The remaining columns of Table 5 are useful in the determination of deflections and stresses. The method of computing the deflections may be explained by reference to Fig. 16. Because of the bending in the voussoir, AC, the tangent to the center line at C will be deflected from its original direction through a small angle, the value of which is,

$$\Delta \beta_B = \sum_C^A \Delta s \frac{M}{E I} \quad \ldots\ldots (40)$$

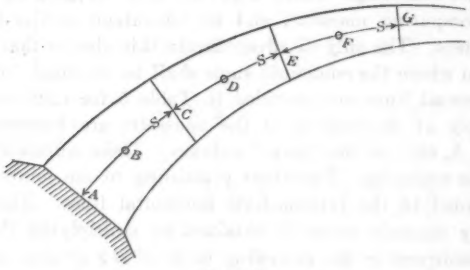

FIG. 16.

If the values of M and I as computed in Table 5 for the point, B, are taken as the average values for the voussoirs, Equation (40) may be written thus:

$$\Delta \beta_B = s \frac{M_B}{E I_B} \quad \ldots\ldots (41)$$

This bending will continue through the succeeding voussoirs, the increments for the voussoirs with centers at D and F being, respectively,

$$\Delta \beta_B = s \frac{M_D}{E I_D} \quad \ldots\ldots (42)$$

$$\Delta \beta_F = s \frac{M_F}{E I_F} \quad \ldots\ldots (43)$$

The total angular movement at E, the upper end of Voussoir D, is, therefore,

$$\beta_E = s \frac{M_B}{E I_B} + s \frac{M_D}{E I_D} = \sum_E^A s \frac{M}{E I} \quad \ldots\ldots (44)$$

The movement at G is, likewise,

$$\beta_E = s \frac{M_B}{E I_B} + s \frac{M_D}{E I_D} + s \frac{M_F}{E I_F} = \sum_F^A s \frac{M}{E I} \quad \ldots\ldots (45)$$

The average angular movement for any voussoir may be taken as the angular movement at its center. Therefore, in Fig. 16, the summations should

be made to the points, B, D, F, etc., if the average angles are desired. In each case, such a summation will include only one-half the last $\Delta\beta$. For this reason half values of $\frac{M}{I}$, Column (33), is obtained by adding each half value in Column (32) twice, reading the summation between the first and last additions of each half value. The factor, s, is omitted from Column (33), to be introduced later, in Column (34). This is possible only when s is constant throughout the length over which the summation is made. The modulus of elasticity, E, is likewise omitted from this column.

These angular movements cause the points along the arch center to be displaced in space. The point, C, will move with reference to A, a distance,

$$\delta_C = \frac{1}{2}\, s\, \beta_C = \frac{1}{2}\, s^2\, \frac{M_B}{E\, I_B} \qquad (46)$$

The direction of motion will be approximately parallel to the radius at B.

The point, E, will move with reference to C, approximately parallel to the radius at D, a distance,

$$\delta_E = s\, \beta_C + \frac{1}{2}\, s\, \Delta\beta_D = \sum_D^A s^2\, \frac{M}{E\, I} \qquad (47)$$

It should be noted that δ_E is the movement of E with reference to C. The movement of E with respect to A will be the geometric sum of δ_C and δ_E. This sum can be most readily obtained by resolving δ_C and δ_E into horizontal and vertical components and adding the components. In the example, Table 5, the individual moments are shown in Column (34). The vertical components of these individual movements are obtained by multiplying each term in Column (34) by the corresponding value of cos α, as recorded in Column (6). The individual components are not recorded in Table 5. The summed products are shown in Column (35); the multiplications are made successively, beginning at the bottom of the column, without clearing the computing machine; and the total is recorded after each operation a half space above the last factors used. The values shown in Column (35) are the vertical deflections of the upper ends of the voussoirs, due to bending only. The horizontal deflections, due to bending, shown in Column (40) are computed in the same way, substituting — sin α for cos α. The minus sign is required in Column (40) for the left half of the arch because of the negative horizontal movement.

In addition to the deflections caused by bending, points on the arch center line are displaced due to changes in length of the voussoirs. The amount that each voussoir will shorten, due to thrust, is shown in Column (36) and the net shortening due to thrust and temperature is shown in Column (37). The values in Column (37) multiplied by — sin α and summed, from the abutment toward the crown, as shown in Column (38), give the vertical deflections due to change in length of the arch center line. In the same way, using — cos α, the horizontal deflections are derived, as shown in Column (41). The negative cosines are required because a positive thrust and a negative temperature change produce negative horizontal and vertical deflections for the left half of the arch.

The total y-deflections, obtained by adding Columns (35) and (38), are shown in Column (39); and the total x-movements, the sum of Columns (40) and (41), are shown in Column (42). These component deflections give the complete computed displacement, in space, of points at the ends of the voussoirs, as C, E, and G, Fig. 16. The resultant radial deflections may be obtained as indicated in Column (43).

The eccentricity, Column (44), is obtained by dividing the moment by the thrust, and is required only when the line of pressure is to be platted, or the portion of the section subject to tension determined. Columns (45) and (46) show, respectively, the average direct pressure and the unit fiber stress due to moment. Both columns are divided by 144 to convert the stresses into pounds per square inch.

The final maximum and minimum fiber stresses are shown in the two columns, which are derived from Columns (45) and (46) as indicated. This numerical example refers to a symmetrical case. Table 5 may be extended to unsymmetrical arch dams by inserting the following columns: (7a), $\frac{\sin \alpha}{T}$; (8a), $\frac{\sin^2 \alpha}{T}$; (9a), $\frac{\sin \alpha \cos \alpha}{T}$; (13$a$), $\frac{x}{I}$; (14a), $\frac{x^2}{I}$; (15a), $\frac{x\,y}{I}$; (25a), $\frac{M_L\, x}{I}$; (26a), $\frac{H_L \sin \alpha}{T}$; (28$a$), $V_C\, x$; and (30a), $V_C \sin \alpha$.

The constants required in the computations may be determined as shown in Table 6, the subscripts, L, denoting the left half of the arch, and R, denoting the right half. Use arch data values, as follows: For the left half: $s = 29.176785$ ft.; $l = 260.992$ ft.; R (up stream) $= 405.00$ ft.; and $t = -6°$. For the right half: $s = 29.176785$ ft.; $l = 260.992$ ft.; R (crown) $= 360.8$ ft.; $c = 0.0000056$; and $E = 288\,000\,000$. Then,

$$c\,t\,E = 0.0000056 \times 288\,000\,000 \times 6 = 9\,676.8$$

$$\frac{c\,t\,E\,l_L}{s} = \frac{0.0000056 \times 6 \times 288\,000\,000 \times 260.992}{29.176785} = -86\,560.87$$

$$\frac{s^2}{E} = \frac{29.176785^2}{288\,000\,000} = 0.0000029558$$

$$\frac{s}{E} = \frac{29.176785}{288\,000\,000} = 0.0000001013$$

$$c\,t\,s = 0.0000056 \times 6 \times 29.176785 = 0.00098$$

Use Table 6 to solve the following simplified equations:

$$A_1\, M_C + B_1\, H_C + C_1\, V_C = D_1 \quad \ldots\ldots (48)$$

$$C_1\, M_C + B_2\, H_C + C_2\, V_C = D_2 \quad \ldots\ldots (49)$$

$$B_1\, M_C + B_3\, H_C + B_2\, V_C = D_3 \quad \ldots\ldots (50)$$

Example.—

By Equation (48):

$$0.00184206\, M_C + 0.052223537\, H_C + 0 = 41\,457.477$$

By Equation (50):

$$0.052223537\, M_C + 3.30428902\, H_C + 0 = 2\,277\,028.44$$

TABLE 6.—COMPUTATIONS FOR CONSTANTS, GIBSON DAM.

$\Sigma_L \frac{s}{I}$	0.00184206	$\Sigma_L \frac{M_L x s}{I}$	
$+\Sigma_R \frac{s}{I}$		$-\Sigma_R \frac{M_R x s}{I}$	
$= A_1$	0.00184206	$+\Sigma_L \frac{H_L s \sin \alpha}{T}$	
$\Sigma_L \frac{s y}{I}$	0.052223537	$-\Sigma_R \frac{H_R s \sin \alpha}{T}$	
$+\Sigma_R \frac{s y}{I}$		$-c t E (h_L - h_R)$	
$= B_1$	0.052223537	$= D_2$	
$\Sigma_L \frac{s x}{I}$		$\Sigma_L \frac{s y^2}{I}$	3.101191720
$-\Sigma_R \frac{s x}{I}$		$+\Sigma_R \frac{s y^2}{I}$	
$= C_1$		$+\Sigma_L \frac{s \cos^2 \alpha}{T}$	0.203097293
$\Sigma_L \frac{M_L s}{I}$	41 457.477	$+\Sigma_R \frac{s \cos^2 \alpha}{T}$	
$+\Sigma_R \frac{M_R s}{I}$		$= B_2$	3 304 289.02
$= D_1$	41 457.477	$\Sigma_L \frac{M_L y s}{I}$	2 379 526.35
$\Sigma_L \frac{x y s}{I}$		$+\Sigma_R \frac{M_R s y}{I}$	
$-\Sigma_R \frac{x y s}{I}$		$-\Sigma_L \frac{s H_L \cos \alpha}{T}$	−15 937.04
$-\Sigma_L \frac{s \sin \alpha \cos \alpha}{T}$		$-\Sigma_R \frac{s H_R \cos \alpha}{T}$	
$+\Sigma_R \frac{s \sin \alpha \cos \alpha}{T}$		$+\frac{c t E (l_L + l_R)}{s}$	−86 560.87
$= B_3$		$= D_3$	2 277 028.44
$\Sigma_L \frac{s x^2}{I}$		$H_C = 604.086$	
$+\Sigma_R \frac{s x^2}{I}$		$M_C = +5\,379.826$	
$+\Sigma_L \frac{s \sin^2 \alpha}{T}$		$V_C = 0$	
$+\Sigma_R \frac{s \sin^2 \alpha}{T}$			
$= C_2$			

Solving simultaneously and eliminating M_C,

$$H_C = 604\,086 \text{ lb.}$$

and,

$$M_C = 5\,379\,826 \text{ ft-lb.}$$

The constants given in Table 6 without arithmetical values will be needed in some of the problems encountered.

From Equation (51), the distribution of p can be calculated. The result is given in Table 7.

Equation (51) is not only valid for a load, p, applied very near the abutment, as p in Fig. 17(a), but it gives directly the ratio of the shear forces in two sections, one horizontal and the other vertical, through a point of the foundation. (See Fig. 17(b).) The shear transmitted to Point A by the cantilever, $A\,B$, and that transmitted to the same point by the arch, $A\,C$, that is, the total cantilever and arch shear, must be in the ratio prescribed by Equation (51), and independent of the elastic characteristics of the dam.

Alfred Stucky* has investigated the Jogne Arch Dam and has determined the division of the water load both with and without temperature variations. Fig. 17(c) shows a horizontal section through the dam at Elevation 790. Stucky determined the load distribution (no temperature variation) in the points marked O, I, II, III, and IV. Point O is the crown, and Point IV is practically at the abutment. In Fig. 17(d) the center line of the arch is developed and the load is divided in the same manner as in the authors' diagrams. The values as given by Stucky are shown in Table 8.

Point IV at the abutment, having a "50-50" division, corresponds to an angle, ϕ, of 45° (Fig. 17(a)).

At Elevation 770, Point II is practically at the abutment and there the arch takes only about 8% of the water load; at Elevation 760, Point I lies at the abutment and there the arch takes 66% of the water load. The load taken by the arch at the abutment is not constantly zero, as assumed by the authors, but varies between wide limits, depending on the slope at each particular point.

TABLE 7.—Distribution of p in Fig. 17(a), Between Arch and Cantilever.

Angle, ϕ, in degrees.	Percentage of Total Load, p.	
	Arch.	Cantilever.
0	0	100
30	36.6	63.4
45	50.0	50.0
60	63.4	36.6
90	100.0	0

The authors have made an attempt to deal with the secondary arch, see Figs. 10, 11, and 15. This leads to difficulties because, in Fig. 15, the angles, $G\,F\,R$, $E\,D\,S$, etc., are not right angles. The ordinary bending theory assumes the sections to be normal to the center line and leads to incorrect results when it is applied to oblique sections.† These uncertainties and inaccuracies may have a considerable influence on the bending moments and deformations and involve the whole calculation in serious difficulties, especially in thick arches where the assistance of the cantilever would be appreciable.

* "Etude sur les barrages arqués," Extrait du *Bulletin technique de la Suisse Romande*, 1922, F. Rouge & Cie., Lausanne.

† "Stresses in Multiple-Arch Dams," by B. F. Jakobsen, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXXVII (1924), p. 289; see, also, discussion by William Cain, M. Am. Soc. C. E., p. 315.

TABLE 8.—DISTRIBUTION OF LOAD BETWEEN ARCHES AND CANTILEVERS IN LA JOGNE DAM AT ELEVATION 790.

Points.	PERCENTAGE OF TOTAL LOAD.	
	Arch.	Cantilever.
Crown	80	20
I	86	14
II	56	44
III	60	40
IV	50	50

The mathematics seem somewhat laborious, but each individual will no doubt work out his own methods of computations along his own particular lines. The fact that the authors have dropped the usual assumption, that the arch is symmetrical, materially complicates the work. Of course, this assumption is, strictly speaking, always incorrect, but it does materially simplify the calculations. Judging from Fig. 5, in which the section is not particularly symmetrical and the loading is practically so, the assumption that the arches are loaded symmetrically does not lead one far astray, except perhaps in unusual cases.

GEORGE R. RICH,* ASSOC. M. AM. SOC. C. E. (by letter).—This excellent paper contributes materially toward a satisfactory evaluation of the many pertinent factors affecting the stresses in an arched dam that has been carefully pressure-grouted near the time of minimum temperature to insure the permanent simultaneous action of both horizontal and vertical elements.

For the short thick arches in the lower sections of the dam, the writer believes that the deflections caused by shearing stresses should not be neglected. He has attempted to show in the following analysis that this effect may be readily included with very little additional labor.

The original notation of the authors has been preserved, with one addition:

Q = the sum of the components of all external forces between the point and the crown, perpendicular to the axis at a given point.

In the formulas given herewith, a subscript, L, refers to the left of the crown and a subscript, R, refers to the right. Table 9, which gives a classification of the various deflection components, is of assistance in writing the basic crown deformations. In this table, E is the modulus for direct stress in all cases. For a rectangular section with parabolic shear distribution, the shear deflection,

$$\Delta = \frac{6}{5} \int \frac{Q\, q\, d\, s}{E_s\, T}$$

in which E_s is the shear modulus. For concrete,

$$E_s = \frac{E}{2.4}, \text{ or } \Delta = 2.88 \int \frac{Q\, q\, d\, s}{E\, T}$$

* Engr., Hydr. Div., Stone & Webster Eng. Corporation, Boston, Mass.

From Equation (51), the distribution of p can be calculated. The result is given in Table 7.

Equation (51) is not only valid for a load, p, applied very near the abutment, as p in Fig. 17(a), but it gives directly the ratio of the shear forces in two sections, one horizontal and the other vertical, through a point of the foundation. (See Fig. 17(b).) The shear transmitted to Point A by the cantilever, $A\,B$, and that transmitted to the same point by the arch, $A\,C$, that is, the total cantilever and arch shear, must be in the ratio prescribed by Equation (51), and independent of the elastic characteristics of the dam.

Alfred Stucky* has investigated the Jogne Arch Dam and has determined the division of the water load both with and without temperature variations. Fig. 17(c) shows a horizontal section through the dam at Elevation 790. Stucky determined the load distribution (no temperature variation) in the points marked O, I, II, III, and IV. Point O is the crown, and Point IV is practically at the abutment. In Fig. 17(d) the center line of the arch is developed and the load is divided in the same manner as in the authors' diagrams. The values as given by Stucky are shown in Table 8.

Point IV at the abutment, having a "50-50" division, corresponds to an angle, ϕ, of 45° (Fig. 17(a)).

At Elevation 770, Point II is practically at the abutment and there the arch takes only about 8% of the water load; at Elevation 760, Point I lies at the abutment and there the arch takes 66% of the water load. The load taken by the arch at the abutment is not constantly zero, as assumed by the authors, but varies between wide limits, depending on the slope at each particular point.

TABLE 7.—Distribution of p in Fig. 17(a), Between Arch and Cantilever.

Angle, ϕ, in degrees.	Percentage of Total Load, p.	
	Arch.	Cantilever.
0	0	100
30	36.6	63.4
45	50.0	50.0
60	63.4	36.6
90	100.0	0

The authors have made an attempt to deal with the secondary arch, see Figs. 10, 11, and 15. This leads to difficulties because, in Fig. 15, the angles, $G\,F\,R$, $E\,D\,S$, etc., are not right angles. The ordinary bending theory assumes the sections to be normal to the center line and leads to incorrect results when it is applied to oblique sections.† These uncertainties and inaccuracies may have a considerable influence on the bending moments and deformations and involve the whole calculation in serious difficulties, especially in thick arches where the assistance of the cantilever would be appreciable.

* "Etude sur les barrages arqués," Extrait du *Bulletin technique de la Suisse Romande*, 1922, F. Rouge & Cie., Lausanne.

† "Stresses in Multiple-Arch Dams," by B. F. Jakobsen, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXXVII (1924), p. 289; see, also, discussion by William Cain, M. Am. Soc. C. E., p. 315.

TABLE 8.—Distribution of Load Between Arches and Cantilevers in La Jogne Dam at Elevation 790.

Points.	Percentage of Total Load.	
	Arch.	Cantilever.
Crown	80	20
I	86	14
II	56	44
III	60	40
IV	50	50

The mathematics seem somewhat laborious, but each individual will no doubt work out his own methods of computations along his own particular lines. The fact that the authors have dropped the usual assumption, that the arch is symmetrical, materially complicates the work. Of course, this assumption is, strictly speaking, always incorrect, but it does materially simplify the calculations. Judging from Fig. 5, in which the section is not particularly symmetrical and the loading is practically so, the assumption that the arches are loaded symmetrically does not lead one far astray, except perhaps in unusual cases.

George R. Rich,* Assoc. M. Am. Soc. C. E. (by letter).—This excellent paper contributes materially toward a satisfactory evaluation of the many pertinent factors affecting the stresses in an arched dam that has been carefully pressure-grouted near the time of minimum temperature to insure the permanent simultaneous action of both horizontal and vertical elements.

For the short thick arches in the lower sections of the dam, the writer believes that the deflections caused by shearing stresses should not be neglected. He has attempted to show in the following analysis that this effect may be readily included with very little additional labor.

The original notation of the authors has been preserved, with one addition:

Q = the sum of the components of all external forces between the point and the crown, perpendicular to the axis at a given point.

In the formulas given herewith, a subscript, L, refers to the left of the crown and a subscript, R, refers to the right. Table 9, which gives a classification of the various deflection components, is of assistance in writing the basic crown deformations. In this table, E is the modulus for direct stress in all cases. For a rectangular section with parabolic shear distribution, the shear deflection,

$$\Delta = \frac{6}{5} \int \frac{Q\, q\, d\, s}{E_s\, T}$$

in which E_s is the shear modulus. For concrete,

$$E_s = \frac{E}{2.4}, \text{ or } \Delta = 2.88 \int \frac{Q\, q\, d\, s}{E\, T}$$

* Engr., Hydr. Div., Stone & Webster Eng. Corporation, Boston, Mass.

in which, q is the shear on the differential element, when a load of 1 lb. is applied to the structure at the point where, and in the direction which, the deflection is desired.*

TABLE 9.—CLASSIFICATION OF DEFLECTION FACTORS.

Bending.	Shear.	Direct stress.
$\Delta\phi = \sum \frac{M\,s}{E\,I}$	$\Delta\phi = 0$	$\Delta\phi = 0$
$\Delta h = \sum \frac{M\,y\,s}{E\,I}$	$\Delta h = 2.88 \sum \frac{Q\,s \sin\alpha}{E\,T}$	$\Delta h = \sum \frac{H\,s \cos\alpha}{E\,T}$
$\Delta v = \sum \frac{M\,x\,s}{E\,I}$	$\Delta v = 2.88 \sum \frac{Q\,s \cos\alpha}{E\,T}$	$\Delta v = \sum \frac{H\,s \sin\alpha}{E\,T}$

Referring to Fig. 14 and Table 9, the motion clockwise of the crown section, left half, equals the motion clockwise of the crown section, right half:

$$+\sum_L \frac{M_L\,s}{E\,I} - \sum_L \frac{V_C\,x\,s}{E\,I} - \sum_L \frac{H_C\,y\,s}{E\,I} - \sum_L \frac{M_C\,s}{E\,I} = -\sum_R \frac{M_R\,s}{E\,I} + \sum_R \frac{M_C\,s}{E\,I} - \sum_R \frac{V_C\,x\,s}{E\,I} + \sum_R \frac{H_C\,y\,s}{E\,I} \quad \ldots(52)$$

The downward motion of the crown section, left half, equals the downward motion of the crown section, right half:

$$\begin{aligned} &+\sum_L \frac{M_L\,x\,s}{E\,I} - \sum_L \frac{V_C\,x^2\,s}{E\,I} - \sum_L \frac{H_C\,x\,y\,s}{E\,I} - \sum_L \frac{M_C\,x\,s}{E\,I} \\ &+\sum_L \frac{H_L\,s \sin\alpha}{E\,T} - \sum_L \frac{V_C\,s \sin^2\alpha}{E\,T} + \sum_L \frac{H_C\,s \sin\alpha \cos\alpha}{E\,T} \\ &-h_L\,c\,t + 2.88 \sum_L \frac{Q_L\,s \cos\alpha}{E\,T} - 2.88 \sum_L \frac{V_C\,s \cos^2\alpha}{E\,T} \\ &-2.88 \sum_L \frac{H_C\,s \sin\alpha \cos\alpha}{E\,T} = +\sum_R \frac{M_R\,x\,s}{E\,I} + \sum_R \frac{V_C\,x^2\,s}{E\,I} \\ &-\sum_R \frac{H_C\,x\,y\,s}{E\,I} - \sum_R \frac{M_C\,x\,s}{E\,I} + \sum_R \frac{H_R\,s \sin\alpha}{E\,T} + \sum_R \frac{V_C\,s \sin^2\alpha}{E\,T} \\ &+\sum_R \frac{H_C\,s \sin\alpha \cos\alpha}{E\,T} - h_R\,c\,t + 2.88 \sum_R \frac{Q_R\,s \cos\alpha}{E\,T} \\ &+2.88 \sum_R \frac{V_C\,s \cos^2\alpha}{E\,T} - 2.88 \sum_R \frac{H_C\,s \sin\alpha \cos\alpha}{E\,T} \quad \ldots\ldots(53) \end{aligned}$$

The motion to the right of the crown section, left half, equals the motion to the right of the crown section, right half:

$$\begin{aligned} &+\sum_L \frac{M_L\,y\,s}{E\,I} - \sum_L \frac{V_C\,x\,y\,s}{E\,I} - \sum_L \frac{H_C\,y^2\,s}{E\,I} - \sum_L \frac{M_C\,y\,s}{E\,I} \\ &-\sum_L \frac{H_L\,s \cos\alpha}{E\,T} + \sum_L \frac{V_C\,s \sin\alpha \cos\alpha}{E\,T} - \sum_L \frac{H_C\,s \cos^2\alpha}{E\,T} \end{aligned}$$

* "Deflections and Statically Indeterminate Stresses," by C. W. Hudson, M. Am. Soc. C. E.

$$+ l_L c t + 2.88 \sum_L \frac{Q_L s \sin \alpha}{E T} - 2.88 \sum_L \frac{V_C s \sin \alpha \cos \alpha}{E T}$$
$$- 2.88 \sum_L \frac{H_C s \sin^2 \alpha}{E T} = - \sum_R \frac{M_R y s}{E I} - \sum_R \frac{V_C x y s}{E I}$$
$$+ \sum_R \frac{H_C y^2 s}{E I} + \sum_R \frac{M_C y s}{E I} + \sum_R \frac{H_R s \cos \alpha}{E T} + \sum_R \frac{V_C s \sin \alpha \cos \alpha}{E T}$$
$$+ \sum_R \frac{H_C s \cos^2 \alpha}{E T} - l_R c t - 2.88 \sum_R \frac{Q_R s \sin \alpha}{E T}$$
$$- 2.88 \sum_R \frac{V_C s \sin \alpha \cos \alpha}{E T} + 2.88 \sum_R \frac{H_C s \sin^2 \alpha}{E T} \ldots\ldots (54)$$

The terms involving shear deformations can be easily identified by the coefficient, 2.88. By transposing, the simultaneous equations for the crown forces are obtained. In setting up numerical equations, the terms involving $\sum \frac{s \sin \alpha \cos \alpha}{T}$ and $2.88 \sum \frac{s \sin \alpha \cos \alpha}{T}$, may be combined; in this derivation they have been left separate to show, by comparison with the authors' original, what terms represent shear deformations:

$$M_C \left(\sum_L \frac{s}{I} + \sum_R \frac{s}{I} \right) + H_C \left(\sum_L \frac{s y}{I} + \sum_R \frac{s y}{I} \right)$$
$$+ V_C \left(\sum_L \frac{s x}{I} - \sum_R \frac{s x}{I} \right) = \sum_L \frac{M_L s}{I} + \sum_R \frac{M_R s}{I} \ldots\ldots (55)$$

$$M_C \left(\sum_L \frac{x s}{I} - \sum_R \frac{x s}{I} \right) + H_C \left(\sum_L \frac{x y s}{I} - \sum_R \frac{x y s}{I} \right.$$
$$- \sum_L \frac{s \sin \alpha \cos \alpha}{T} + \sum_R \frac{s \sin \alpha \cos \alpha}{T} + 2.88 \sum_L \frac{s \sin \alpha \cos \alpha}{T}$$
$$\left. - 2.88 \sum_R \frac{s \sin \alpha \cos \alpha}{T} \right) + V_C \left(\sum_L \frac{x^2 s}{I} + \sum_R \frac{x^2 s}{I} + \sum_L \frac{s \sin^2 \alpha}{T} \right.$$
$$\left. + \sum_R \frac{s \sin^2 \alpha}{T} + 2.88 \sum_L \frac{s \cos^2 \alpha}{T} + 2.88 \sum_R \frac{s \cos^2 \alpha}{T} \right)$$
$$= \sum_L \frac{M_L x s}{I} - \sum_R \frac{M_R x s}{I} + \sum_L \frac{H_L s \sin \alpha}{T} - \sum_R \frac{H_R s \sin \alpha}{T}$$
$$+ 2.88 \sum_L \frac{Q_L s \cos \alpha}{T} - 2.88 \sum_R \frac{Q_R s \cos \alpha}{T} - c t E (h_L - h_R) \ldots (56)$$

$$M_C \left(\sum_L \frac{y s}{I} + \sum_R \frac{y s}{I} \right) + H_C \left(\sum_L \frac{y^2 s}{I} + \sum_R \frac{y^2 s}{I} \right.$$
$$+ \sum_L \frac{s \cos^2 \alpha}{T} + \sum_R \frac{s \cos^2 \alpha}{T} + 2.88 \sum_L \frac{s \sin^2 \alpha}{T}$$
$$\left. + 2.88 \sum_R \frac{s \sin^2 \alpha}{T} \right) + V_C \left(\sum_L \frac{x y s}{I} - \sum_R \frac{x y s}{I} \right.$$
$$- \sum_L \frac{s \sin \alpha \cos \alpha}{T} + \sum_R \frac{s \sin \alpha \cos \alpha}{T} + 2.88 \sum_L \frac{s \sin \alpha \cos \alpha}{T}$$
$$\left. - 2.88 \sum_R \frac{s \sin \alpha \cos \alpha}{T} \right) = \sum_L \frac{M_L y s}{I} + \sum_R \frac{M_R y s}{I}$$

$$-\sum_L \frac{H_L\, s \cos \alpha}{T} - \sum_R \frac{H_R\, s \cos \alpha}{T} + 2.88 \sum_L \frac{Q_L\, s \sin \alpha}{T}$$
$$+ 2.88 \sum_R \frac{Q_R\, s \sin \alpha}{T} + c\, t\, E\, (l_L + l_R) \ldots\ldots\ldots\ldots (57)$$

It can be readily seen that these equations are identical with those of the authors, with the exception of the terms added for shear movements.

After working through the various differential deflections, the writer has some appreciation of the difficulty of avoiding errors in algebraic sign in spite of careful checking. For this reason he would ask that Mr. Howell verify these additions to Equations (27), (28), and (29), and then compute the division of load for a thick, short-radius arch (such as the Gibson Dam at Elevation 4 530), first, considering shear deflections, and, second, neglecting shear deflections so that a comparison may be available to the profession.

In computing the movements of thick cantilever elements, the same attention should be given to each of the deflection groups: For bending, $\int \frac{M\, m\, d\, s}{E\, I}$; and for shear, $\frac{6}{5} \int \frac{Q\, q\, d\, s}{E_s\, A}$, in which, m and q are the moment and shear, respectively, on the differential element caused by a load of 1 lb. acting on the structure at the point where, and in the direction which, the deflection is desired. M and Q are the general expressions for moment and shear, respectively, on any differential element due to the external loads. The writer would feel indebted to Mr. Howell for incorporating this deformation in the numerical example requested.

The writer hopes that the final closure of the dam by pressure-grouting will eventually disclose great latent possibilities for fixing the limiting stresses in the arches under the assumed range of temperature. This case appears somewhat analogous to the erection of a spandrel-braced steel arch bridge. After starting erection by cantilevering from back-stays, the bottom chord of the bridge is closed at the crown as a three-hinged arch for dead load; then hydraulic jacks are inserted into the top chord; stresses are induced corresponding to the two-hinged condition for dead and live load, and the final riveting is done under the two-hinged condition. A prohibitive length of time is required for the total dissipation of the chemical heat liberated by the setting of large masses of concrete in an arch dam; in addition, a fast construction schedule may make a closure in the summer months imperative. Under such conditions, it would still be at least theoretically possible to preclude vertical shrinkage cracks by inducing compressive stresses of magnitude sufficient to cancel the tensions caused later by shrinkage. The effect of a rise in temperature in raising compressive stresses would also have to be carefully anticipated; but the compressive stresses seldom reach their limiting values, and a given increase in unit compressive stress is much more readily accommodated than the occurrence of a tension of equal magnitude.

The adequate drainage both of the dam proper and the foundation rock, and also the insertion of grout pipes in the bed-rock near the heel, prime requirements of every modern design, amply warrant the uplift assumption

used by the authors for the cantilever elements, namely, full head-water pressure at the heel diminishing to zero at the toe, and acting over one-half the base section.

The present indications are that foundation deformations do have an appreciable effect on the division of load between arches and cantilevers. Similarly, when the phenomena of water-soakage are more clearly defined, commensurate refinements can be applied to this valuable method of attack.

The writer is thoroughly sympathetic with the author's "Conclusions". It is to be hoped that ultimately a rational theory embracing all the really pertinent elements can be made available in the form of curves; in the pursuit of this ultimate object, the paper deserves careful study.

C. A. P. TURNER,* M. AM. SOC. C. E. (by letter).—The authors are entitled to appreciation of the profession for their laborious effort to place the design of curved dams on a more rational basis. The increase in strength and economy of the curved vertical dam over that of the straight gravity type is little appreciated because of a fundamental misconception regarding its mode of resistance. In elementary mechanics the student is taught that the applied moment of an external force is the product of that force times its lever arm. Applied moment is an abstract quantity, balanced either by internal bending or twisting resistance, depending on the manner of support, restraint, and the form of the resisting section.

In many cases the internal resistance of the member considered is entirely bending resistance. In others, such as the shaft, the internal resistance is almost wholly that of twisting resistance. In the square plate supported on four sides the resistance to flexure is half bending and half twisting. A curved dam, because it is fixed at the base so that it cannot deflect down stream, tends to oppose water pressure predominantly by twisting, rather than by arch action. The applied external force consists of the normal hydraulic pressure that increases uniformly, downward from the top, along the vertical up-stream face.

The horizontal reaction between the dam and its base is represented by the sum of the horizontal down-stream components of the hydraulic pressures on a vertical up-stream face. The components of these pressures parallel to the chord of the arc of the dam furnish the external reactions to the internal ring compression in the arched masonry or concrete of the dam. The intensity of these ring stresses for an economic design are small relative to those developed by the twisting shears.

A curved dam may be considered analogous to a vertical curved plate rigidly restrained at the base and ends, and subject to the pressure of parallel forces. These tend to twist the plate along its length, causing a maximum rotation at the deepest part. As the action is a twisting one there must be both a positive and a negative twisting deformation. Were the plate free at the base, restrained at its end, and then twisted from the center, the points of equilibrium would be at the quarter-points and a vertical plane at a quarter-point would be the locus of equilibrium of the positive and negative

* Cons. Engr., Minneapolis, Minn.

twisting shears. However, because the arch is rigid both at its base and at its ends at the quarter-point of the free crest, the direction of the plane of equilibrium will rotate from the vertical to an angle of 45° with the horizontal projection of the crest of the dam.

In a plate deformed by twisting and bending, contours of curvature may be developed by graphic methods as applied to uniform elastic plates supported, restrained, and bent in any manner as previously outlined by the writer. To apply the method to the curved plate of the dam, treat the down-stream component of the pressures separately as these component pressures and the reactions thereto constitute the applied torsional forces.

The first step in this analysis is to fix the travel of the pressure toward its support. This travel of the load or pressure follows a straight line and in the case of plates which are twisted these lines intersect at some given point which may be termed a focus. The plane of equilibrium of the positive and negative twist would represent one of these planes of zero shear and the focus of intersection of the other planes will lie along this line at some point to be determined. Curves of greatest shear will be normal to those of zero shear and the co-ordinated intersections will determine one series of the twisting shear curves. The co-ordinated intersections of the planes of zero shear and of greatest twist will fix another or complement series of twisting shears in the opposite direction from the first set. Symmetrical intersections of the twisting shears will fix the contours of curvature which are principal stress lines at the mid-thickness of the plate.

Because when an element is twisted in one direction on one side of the plate and in the other direction on the other side, equilibrium will occur at mid-length, there are two independent series of shear curve intersections which locate the same principal stress line or contours of curvature.

In Fig. 18, a graphic analysis is offered of an opening between rock walls similar in form to the experimental dam at Stevenson Creek, in which the sides have a slope of 45 degrees. The first problem is to locate the foci of the planes of zero shear representing the transference of the down-stream hydraulic pressure to its reaction at the base. Because the crest of the dam is free to deform in an elastic manner, the length of the projected positive and negative twisting deformation will be equal and the deflection of the crest will pass through zero at the quarter-point of its down-stream projected length. The equilibrium of the positive and negative twisting, that occurs at the mid-thickness on a line at 45° to the crest, will represent one plane of zero shear and the focus of the series or pencil of planes of zero shear will lie somewhere along the length of this line.

The abscissa of the focus for each respective side of the dam about its center line will lie at the same distance from the center as the center of the line of equilibrium of the positive and negative twist, because twisting deformation accumulates along the length of the member twisted and this length fixes the horizontal position of the focus.

It is then necessary to co-ordinate the lines of greatest shear, greatest twist, and zero shear. For this purpose it is generally convenient to use four equal

divisions for the co-ordinates in the positive and negative areas, respectively, treating the twist about the vertical as productive of equal shifts. Equal pressures must be represented by the spacing of the horizontal planes of greatest twist; that is, take the water pressure increasing from zero at the crest to a maximum at the bottom, and the projected crest of the dam will be the first plane of greatest twist. The half depth will represent the second

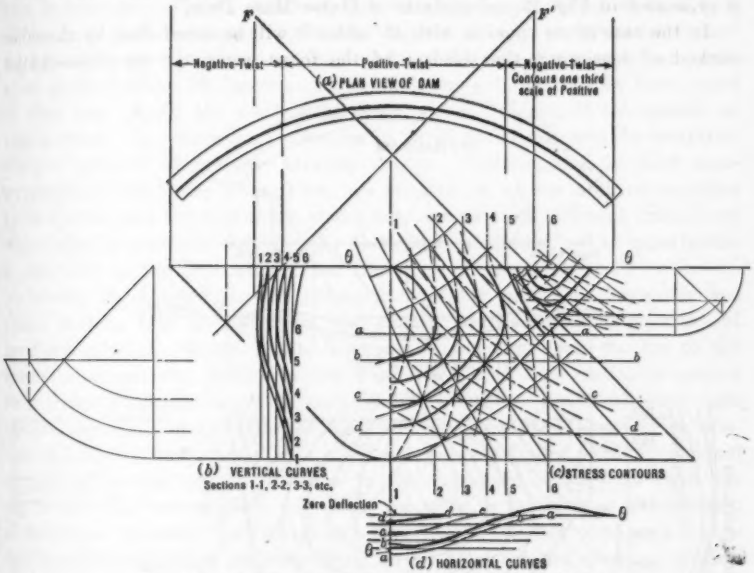

FIG. 18.—GRAPHIC ANALYSIS, STEVENSON CREEK DAM.

plane, and 0.7071 times the depth at the center will represent the third, etc. The vertical areas between the planes of greatest twist will represent equal areas of pressure. Having divided the vertical pressure (for the positive area twisted) into equal pressure areas or parts, it is necessary to co-ordinate the circular curves of greatest shear by dividing the vertical height (from the crest to the base) into four equal parts and drawing circles from the foci as a center to represent the curves of greatest shear. Then the twisting spirals are drawn as shown in Fig. 18 (*b*) and (*d*) and the contours of curvature, Fig. 18(*c*), are located therefrom. The area of negative twist is treated in like manner.

In treating the negative area of curvature from twist by proceeding along like lines the contours would represent a different magnitude of negative deflection. This difference would vary in proportion as the depth of the base of the negative area varies with the depth of the base from the water level in the positive area. On this scale, the deflection curves may be readily co-ordinated and contours interpreted. As applied to the Stevenson Creek Dam, the graphic method develops deflections in striking agreement with those measured.

As the method outlined is general rather than special it must also apply to a different form or shape of opening between rock walls. Its accuracy as a general solution is to be judged by the rationality of the result developed by wide variation in the areas of positive and negative twisting. A rectangular opening between rock walls, nearly as deep or deeper than the width, will give a satisfactory opportunity for the application of the method. This is presented in Fig. 19, an analysis of Horse Mesa Dam.

In the case of an opening with 45° sides it will be noted that, by the same method of reasoning, the abscissa of the focus occurs at the three-eighths

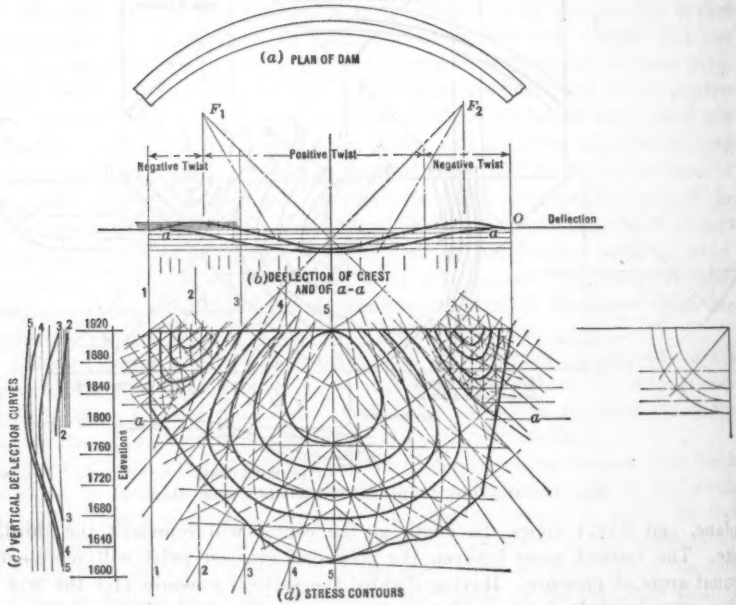

FIG. 19.—GRAPHIC ANALYSIS OF CURVED DAM, HORSE MESA DAM PROFILE.

point of the half chord of the arc. For the rectangular opening it occurs at the quarter-point measured, in each case, from the end of the arc. Thus, the focus shifts horizontally and vertically with the change in the configuration of the opening. In like manner the relative spacing of the planes of greatest twist change, in the positive and negative areas, with the change in configuration of the opening.

It is now possible to compare the contours determined by this method and the exact method of torsional analysis. The authors have assumed that all the load is carried by the horizontal and vertical elements instead of diagonally in the most direct line to the support. The contours in Fig. 9 form a cusp at the vertical center line of the dam. It is a fundamental property of internal elastic stress as stated by Eddy* that the stress on an

* "Graphical Statics," 1878.

ideal plane of division that traverses any given point of a body cannot change suddenly, either as to direction or magnitude, while that plane is turning in any way about the given point. A sudden change can only take place at a surface where there is change in the material.

Because a contour of curvature is identical with the principal stress line at the mid-thickness of the plate, the error embodied in the authors' assumption is apparent. The direction of the contour of zero deflection in Fig. 9 differs from the true contour by convex instead of concave curvature toward the negative area. In direction it differs from the true direction by an angle equal to from 45 to 80 degrees. Accordingly, as principal stress lines, some of them may occupy the position of shear curves. Hence, if interpreted as tensile stress, the strength of concrete in shear resistance may be mistaken for the weakness of concrete in pure tension. Furthermore, in their computations of the Horse Mesa Dam, no recognition of the area of negative twist appears and the formation of the contours is quite different from those which may be correctly developed by the graphical method or by experiment, as was done at the Stevenson Creek Dam.

It may be objected that in presenting graphic methods, the writer has failed to take into consideration the change in the torsional moment of inertia incident to change in the thickness of the dam from the top to the bottom and that any method which would disregard such variation cannot be considered approximate even to a moderate degree. Such criticism fails to take into consideration the fact that the graphic method determines relative not actual deflections; that a fairly wide variation in the torsional moment of inertia (due to increase in the thickness of the dam from the top toward the bottom) does not affect the relative position or the location of deflection contours. For that reason the graphic analysis presents a simple and expeditious manner of determining elastic behavior and a correct method of finding the distribution of the reaction along the length of the dam. With this distribution known it should not be difficult to compute the vertical couples which oppose overturning and the stresses resulting therefrom.

The trial-load method endeavors to treat a torsional phenomenon as if there were a flexural equivalent thereto, to wit, cantilever action and arch action. As the trial-load method depends on distribution of the applied pressure toward the support in vertical and horizontal directions, the resultant of these distributions at each unit of vertical height may readily be determined. It will then be found that the resultants do not follow a regular law of distribution and do not, in the most direct manner, convey the pressure to its support as required by the law of least work.

The twisting moments brought into action by the restraint of the dam at its base and ends takes effect by changing the direction of the pressure and altering the distribution of the reaction thereto along the base of the dam. The effect of any variation in thickness under such action differs wholly from the phenomenon of flexure and the graphic method permits an analytical investigation of the variation in contours of relative deflection to the extent that they are affected by the variation in form and thickness of the structure.

The curves of greatest shear are, of necessity, circles drawn from the foci as centers. The planes of greatest twist are fixed by the intensity of the applied hydraulic pressure such that their spacing, per unit, represents equal pressure areas. The only variation then, that may be effected by a change in the thickness of the dam, is the position of the focus and spacing of the radials of zero shear and the depth of the negative area. Provided the crest of the dam is made of a constant width, the point of inflection (where the positive twisting changes into negative) will remain substantially fixed and the spacing of the radials (because they depend on this locus) remain unchanged even though the lower portion of the dam is increased in thickness.

The deflection is reduced by thickening the dam at the base, but the locus of the contours of equal deflection remains substantially fixed because the co-ordinates that determine the twisting shears remain substantially fixed. An abrupt change in the thickness would affect the accuracy of this generalization. That the function of the curved dam is to oppose twisting moments rather than flexure substantiates the trend of practice toward a more uniform section along the length of the dam than has been practiced in many examples of older construction.

An acceptable elastic analysis must be verified by its elastic deportment. Unfortunately, the elastic behavior of curved dams has been investigated systematically only in the single example of the Stevenson Creek Dam.*

Deflection of the dam is shown graphically in diagonal projection in Fig. 20. A similar diagonal projection of the relative deflections, determined graphically from Fig. 18, may be compared therewith. It will be noted that the contours of negative twist fix a slightly different curvature from that shown in the report of Engineering Foundation, referred to, in that the restraint at the end of the dam manifests itself in an additional point of contraflexure neglected by the draftsman in drawing the curves through measured points on the experimental dam. This does not show a difference in relative deflections where measured, but rather tends to correct an error of interpretation of the measurements made. The curved contour of zero deflection, measured at Stevenson Creek, matches those determined by the graphic method.

Every elastic plate or curved dam, in which sudden jogs in the thickness of the material are avoided, will deflect in a smooth regular curve and contours, whether determined graphically or by computation, and, if rational, should determine smooth consistent flexure curves. The results, by graphic analysis, treating the arch resistance as torsion (Fig. 18), present such curves. The diagonal projection of Fig. 20 may be compared to the Progress Report No. 3, of Engineering Foundation, and the agreement between the relative theoretical and measured deflections is sufficiently close to carry conviction.

In the experiments on the Stevenson Creek Dam, a slight shrinkage crack on the surface was noted, which was observed to close up as the water pressure approached the crest of the dam. This results from the ring stresses by the component of the hydraulic pressure parallel to the chord of the arch.

* See Progress Rept. No. 3, Engineering Foundation, Fig. 2.

The graphic method of analysis permits the simple derivation of the curves of flexure of a plate bent in any manner and subjected to any kind of restraint, manner of support, or applied load. This solution is not always obvious but, when once worked out, the procedure is simple and the method gives the investigator an exact conception of the displacement curves and equi-potential curves, in addition to the curves of shear and principal stress. It permits the development of the contours of distortion which the plane normal section undergoes during the twisting action.

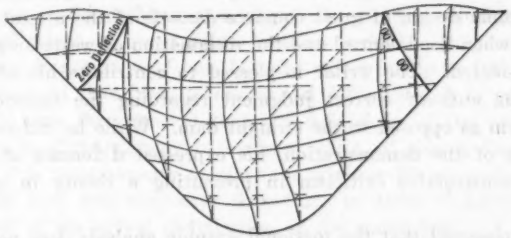

FIG. 20.—DIAGONAL PROJECTION OF CONTOURS OF DEFLECTION, GRAPHIC ANALYSIS, STEVENSON CREEK DAM.

For example, in a round shaft under torsion, half the end area is increased in length and half decreased. The entire area within a radius of $0.7071R$ is elongated by the radial compressions and circular tensions developed in the twisting process. If the cross-section is square, the rotational couples about consecutive radii are not equilibrated by uniform intensity of the shearing stress in the normal planes as is the case in the circular shaft. The result is a deformation consisting of alternate octantal bulge and depression which, in a particular octant, is dependent on the direction of the twisting force.

When the shape of the specimen is changed to rectangular, and the long side is one and one-half times the breadth, the alternate bulge and depression occur in alternate quadrants and in the case of a curved dam the area of depression would be the upper down-stream quadrant. The quadrants of bulging distortion would be the upper up-stream and the lower down-stream quadrants. This torsional distortion would manifest itself in a thrust at the base, although the elastic strain therefrom would be relatively slight as measured in the Stevenson Creek experiments.

Torsional analysis of the curved dam indicates that material has been used with lavish wastefulness in the deeper portion of the dam and, in general, it has been thinned down too rapidly toward the ends. A more uniform section is better adapted to resist the torsional stresses and would result in greater strength with less material.

The economic form of the arch in a V-opening is a third degree curve rather than a circular one. Such a structure, to be well proportioned, should cut the cost of the ordinary gravity dam in half allowing a greater degree of safety. This is particularly true when the concrete is properly reinforced,

thereby insuring toughness, dependability, and resistance to temperature stress.

In pure bending, the distance the load is shifted in its travel to the support, multiplied by the weight shifted, equals the applied moment. Therefore, in torsion, the distance the pressure is shifted by twist, times its weight, equals the portion of applied moment balanced thereby when the shift lies in the plane of action of the applied moment. In the straight dam this shift does not lie in the plane of the over-turning moment and therein the curved dam derives its great advantage over the straight dam.

Safe economic design of great concrete dams is of the utmost importance to the public when flood control and the reclamation of waste areas by irrigation are considered. The writer is pleased to contribute his effort in substantiating the authors' correct judgment regarding the superior merit of the curved dam as opposed to the straight dam. While he differs with them in the details of the demonstration, his expressed difference of opinion is intended as constructive criticism in presenting a theory in accord with experiment.

It will be observed that the torsional graphic analysis does not primarily take the curvature of the dam into consideration; that is, it is applicable alike to the straight and curved dam, but it deals only with the torsional effects in distributing the load along the base. Torsion shifts pressure or shear in its transfer to the support. In the curved dam there is a down-stream shift by virtue of curvature, accompanying the longitudinal shift.

In a V-opening the longitudinal shift distributes the reaction at a higher elevation and may thereby reduce the applied moment a substantial amount in either a straight or a curved dam. The shift of the reaction along the base when the dam is curved, has a down-stream component and this adds to the lever arm of the reaction in opposing over-turning moment. The sharper the curvature the greater the arm; also the greater the length of the dam. A balance between the disadvantage of increased cost of additional length and the advantage of greater lever arm and reduced thickness, constitutes the problem of economic design.

In an examination at a school of engineering, if the student should undertake to compute the strength of a shaft by a beam formula, he would be considered deficient in knowledge of the principles of elementary mechanics; and yet many post-graduate engineers, with years of experience, undertake to calculate the torsional problem of the curved dam by the ordinary old beam formula for a gravity type structure unadapted to the conditions of the problem.

In conclusion, the credit for this graphic solution is in a large measure due to the laborious and painstaking effort of Messrs. Howell and Jaquith in their endeavor to reduce curved dam analysis to a rational basis. Interest in their work and an attempt to analyze it led to the writer's first application of graphical principles to the problem under discussion. Ernest McCulloh, M. Am. Soc. C. E., assisted in the preparation of this discussion and furnished many helpful suggestions.

F. H. CONSTANT,* M. AM. SOC. C. E. (by letter).—The authors present four general conclusions to which the writer wishes to add comments, as follows:

1.—If the cantilever acts, the arch elements are not under uniform loading. With this conclusion the writer is in hearty accord. Any treatment of arches which considers only uniform loading is of little practical importance in the solution of arch dams.

2.—Computing proportionate loads and deflections in the two systems at one vertical section only, such as at the crown of the arches, is not sufficient for the solution of the problem as a whole. A complete solution should find coincident deflections at every point in every section. It is clear that the method of trial is not an easy one, but the writer has gone far enough with it to feel convinced that in spite of its real difficulties, it is practical. He found (as did also the authors) that the deflections of the cantilevers are very sensitive to slight changes of loading, while, of course, the stresses are not. Hence, it is not necessary to carry the trials so far as to secure absolute coincidence of arch and cantilever deflections in order to arrive at a close approximation of the actual distribution of the water load.

3.—Rational formulas, for a complete solution, must take account of the shape of the canyon. The trial method does this by using a great many vertical and horizontal sections. It is not possible to do it by direct mathematical means. The rational formulas that have been heretofore devised fit only special and very simple cases, and any attempt to adapt them to some other type of dam section makes them hopelessly complex.

4.—In place of the authors' fourth conclusion the writer presents somewhat simpler formulas for the special (and yet very common) case of the circular dam of constant thickness throughout any horizontal section and symmetrical relative to the vertical section at the crown. Unsymmetrical dams do occur, of course, but on the whole there is a general symmetry about most narrow profiles. The dams cited by the authors are quite symmetrical.

The writer also feels justified in treating the case of arches of constant thickness. The curve of pressure in an arch dam does not deviate from the axis of the arch nearly as much as in arch bridges, and there would not seem to be any necessity for varying the thickness, except possibly at the very end. In an approximate solution of the Stevenson Creek Dam, assuming a parabolic distribution of the water load, and neglecting temperature effects, the writer found that the curve of pressure remained practically within the middle third throughout, at least in the upper 40 ft. of the dam. It would be possible to change the shape of the arches slightly to conform more closely with the curve of pressure, and thus eliminate all but temperature moments. The error thus introduced in using the formulas for circular arches would be negligible.

The following formulas were derived from the principle of work by well-known methods and the derivations will not be given. They have been carefully checked by independent methods and are believed to be correct. For example, the deflections check by Maxwell's principle of reciprocal deflections.

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The following notation (see Fig. 21), has been used. All angles refer to the half arch. They are in radians, when they occur alone; in degrees, when they occur in functions:

ϕ_1 = the central angle between the center and the end of arch;
ϕ = the central angle to the point at which the moment is required;
θ = the central angle to the point of application of the concentrated load, P;
θ_1 = the central angle to the point at which the deflection is required;
D = the radial deflection of the arch at θ_1 due to a pair of symmetrical radial loads, P;
R = the radius of the axis of the arch;
I = the moment of inertia of the arch, with a vertical width of 1 ft.;
t = the constant horizontal thickness of the arch;
M_0 = the bending moment at the crown, produced by a pair of symmetrical radial loads, P;
H_0 = the horizontal thrust at the crown, produced by a pair of symmetrical radial loads, P;
M = the bending moment at a section, under the same loading;
P = the applied radial force, taken as unity. There is a symmetrical force on the other half of the arch (not shown in Fig. 21);
p_1, p_2, etc. = intensities of water pressure on the arch at Points 1, 2, etc.;
E = the modulus of elasticity of concrete. If the same value of E is used for the cantilevers, it may be omitted from the equations in equalizing deflections. The deflections found, then, will be E times the actual deflection;
$A = t$ = area of cross-section of an arch element, in square feet; and,

$$C = \frac{\sin \phi_1}{\phi_1}.$$

$$M_0 = R\,\frac{1 - \cos(\phi_1 - \theta)}{\phi_1} - H_0\,R\left(1 - \frac{\sin \phi_1}{\phi_1}\right) \quad \ldots\ldots\ldots (58)$$

$$H_\theta = \frac{\frac{1}{2}\left[\sin(\phi_1 - \theta)\sin\phi_1 - (\phi_1 - \theta)\sin\theta\right] - C\left[1 - \cos(\phi_1 - \theta)\right]}{\frac{1}{4}\sin 2\phi_1 - 2\,C\sin\phi_1 + \phi_1\left(\frac{1}{2} + C^2\right) + \frac{I\,\phi_1}{A\,R^2}\left(= \frac{t^2\,\phi_1}{12\,R^2}\right)} \quad \ldots\ldots\ldots (59)$$

$$M = M_0 - R\sin(\phi - \theta) - H_0\,R(\cos\phi - C) \quad \ldots\ldots\ldots (60)$$

$$\frac{E\,I}{R^2}\,D = (M_0 + C\,R\,H_0)\left[1 - \cos(\phi_1 - \theta_1)\right] - \frac{R}{2}\left[\left(\phi_1 - \begin{matrix}\theta\\ \theta_1\end{matrix}\right)\cos(\theta - \theta_1)\right.$$
$$\left. - \sin\left(\phi_1 - \begin{matrix}\theta\\ \theta_1\end{matrix}\right)\cos\left(\phi_1 - \left(\begin{matrix}\theta_1\\ \theta\end{matrix}\right)\right)\right] - \frac{R\,H_0}{2}\left[\sin(\phi_1 - \theta_1)\sin\phi_1\right.$$
$$\left. - (\phi_1 - \theta_1)\sin\theta_1\right] \quad \ldots\ldots\ldots (61)$$

In the double value expressions, $\begin{matrix}\theta\\ \theta_1\end{matrix}$ and $\begin{matrix}\theta_1\\ \theta\end{matrix}$, use the upper values if $\theta > \theta_1$; the lower values if $\theta_1 > \theta$.

By varying the angles in Equations (58) to (61) influence values for the four quantities, in all the arches, can be found. The computations should be made in tabular form and they then become quite simple and mechanical. They should be made with a calculating machine, and all results set down to at least six decimal places.

Fig. 23 is a typical deflection influence line. It shows the change in deflection at the crown of the top arch as a pair of radial unit loads move from the crown symmetrically over the left and right halves of the arch, respectively.

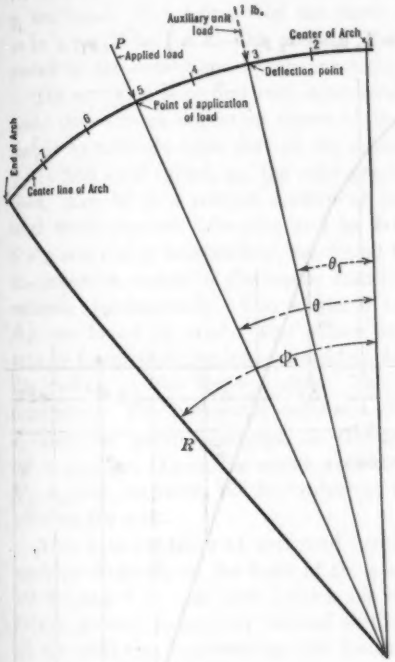


FIG. 21.

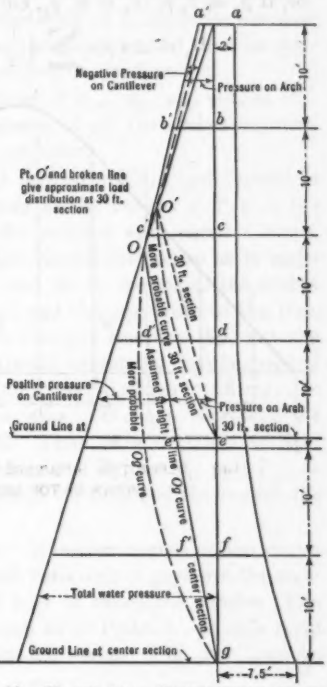


FIG. 22.—WATER LOAD DISTRIBUTION DIAGRAM.

An assumption is now made in regard to the variation in radial water pressure along any horizontal arch. If the intensity of the radial water pressure, p, is uniform along the arch, the deflection at any point would be the area of the corresponding influence line multiplied by p. If p is not uniform, but varies in some assumed manner, as shown in Fig. 23(*b*), the deflection of any arch at any point can be found as follows.

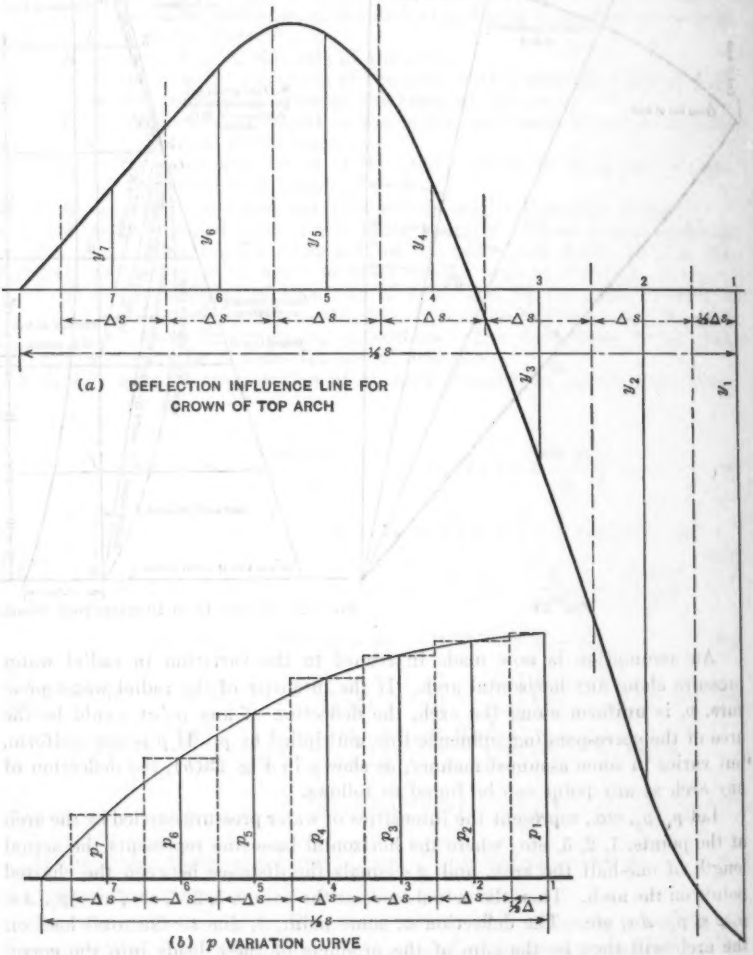
Let p_1, p_2, etc., represent the intensities of water pressure carried by the arch at the points, 1, 2, 3, etc., where the horizontal base line represents the actual length of one-half the arch, and Δs equals the distance between the charted points on the arch. Then the actual loads at the points, 1, 2, 3, etc., are $\frac{1}{2}p_1$, Δs, $p_2 \Delta s$, p_3 Δs, etc. The deflection at some point, 3, due to the total load on the arch will then be the sum of the products of these loads into the corresponding ordinates of the influence line for Point 3 of that particular arch. It is best to express p_2, p_3, etc., as some fraction of p_1. For example, to find

the deflection, D_3, at the point, 3, of an arch, let the ordinates in the influence line at 1, 2, 3, etc., be y_1, y_2, y_3, etc. Then,

$$D_3 = \Delta s \sum \left(\frac{1}{2} p_1 y_1 + p_2 y_2 + \cdots \right)$$

or, if $p_2 = k_2 p_1$, $p_3 = k_3 p_1$, etc.,

$$D_3 = \Delta s \, p_1 \sum \left(\frac{1}{2} y_1 + k_2 y_2 + k_3 y_3 + \cdots \right) = K_3 p_1$$



(a) DEFLECTION INFLUENCE LINE FOR CROWN OF TOP ARCH

(b) p VARIATION CURVE

FIG. 23.

For the assumed distribution and the given deflection influence lines, values of K can be obtained for each point of each arch, and the deflection

of that point will be known as soon as p_1 is found. It might be convenient to construct a complete table of the values of K to include all the charted points, using some notation such as K_{13}, K_{25}, etc., meaning thereby, Point 3 of Arch 1, Point 5 of Arch 2, etc. The values of K_{11}, K_{21}, etc., will give the crown deflections of the several arches as soon as the corresponding values of p_1 are found. For a first trial the curve of variation of p might be assumed to be a parabola, for all the arches. Further modification will then be suggested by the deflections which parabolic distribution produces.

The next step is to find such numerical values of p_{11}, p_{21}, p_{31}, etc., as will cause the vertical deflection curve of the crowns of all the arches approximately to coincide with that of the center cantilever.

To find such values, p_1, the semi-graphical method of Cain and Noetzli is used. Fig. 22 is a vertical section of the dam at the crown; $a\ g'\ g$ is the total water-pressure triangle; and by trial the positive and negative areas, $O\ g'\ g$ and $O\ a'\ a$, respectively, are found by the Noetzli method so as to make the deflection curves of the center cantilever and of the crowns of the arches coincide approximately. The points, O and a', and the curvature of the line, $O\ g$, are found by trial. The writer used a straight line for $O\ g$, but the authors found that $O\ g$ has a considerable outward curvature, thus increasing the loading on the lower arches. The writer's investigation confirms this conclusion. The horizontal ordinates of the area, $g\ O\ a'\ a\ g$, represent the p_1-values, or crown pressures, on the arches. Thus (Fig. 22), $aa' = p_{11}$; $bb' = p_{21}$; etc. Then, the crown arch deflections are, $D_{11} = K_{11}\ p_{11}$; $D_{21} = K_{21}\ p_{21}$; etc., in which, the first subscript indicates the arch and the second, the point on the arch.

After a coincidence of arch and cantilever deflection curves at the crown has been obtained, on the basis of the assumed variation of pressure throughout the length of each arch (which for short may be called the p-curve (Fig. 23(b)), proceed to another vertical section, such as at Point 3. At this point all the ordinates representing the loading on the arches (Fig. 22), namely, aa', bb', etc., must be reduced in the fixed ratio, $\frac{p_3}{p_1} = k_3$. The point, O, will accordingly lie nearer the top, and the lower part of the diagram is cut off to conform to the ground level. The dotted lines in Fig. 22 represent the approximate condition. The cantilever and arch deflection curves are constructed as at the crown in the usual manner. Thus, the deflection curve of the arches at Section 3 can be drawn from $D_{13} = K_{13}\ p_{11}$; $D_{23} = K_{23}\ p_{21}$; etc., in which, $p_{11} = aa'$; $p_{21} = bb'$, etc. (Fig. 22). If the p-curve has been correctly assumed these arch and cantilever deflection curves should be coincident at Section 3. If they are not, within reasonable limits, a new p-variation curve, or a different center section distribution (Fig. 22) will have to be assumed until a fair check is obtained. For accurate results several sections should be tested in this manner.

Instead of a parabolic variation of water pressure on a horizontal arch the authors have used Fig. 5(c), "Final Proportional Load," which shows a nearly uniform distribution over the middle two-thirds of the arch, decreasing

rapidly to zero near the ends. As the area enclosed is greater than that by a parabola having the same center height, the use of such a curve instead of the parabola would result in increased calculated compressive fiber stresses.

The writer is led to believe, as a result of his own investigations on a similar dam, that the proportionate load distribution found by the authors, and shown in their Figs. 5 and 7, must be nearly correct. It is obvious, as experience is acquired in the use of this method, closer first assumptions will be made and the goal reached much more quickly than at present.

The writer wishes to express his appreciation of the valuable assistance given by Joseph B. Reynolds, Professor of Mathematics, Lehigh University, and D. B. Sloan, Jun. Am. Soc. C. E. Professor Reynolds checked all the mathematical work and made valuable suggestions which led to the present compact form of the working equations.

F. W. HANNA,* M. AM. SOC. C. E. (by letter).—The trial and error method of analysis used by the authors conforms to the customary mathematical practice of solving involved equations by successive approximations. The method, therefore, will produce reliable results, if the fundamental equations are based on sound assumptions. However, in any mathematical problem, it is more satisfactory to proceed by direct methods if it is economical and feasible to do so.

The authors have given a comprehensive and clear development of stress and deflection formulas comprising the effects of bending moment and thrust, but have omitted the very important effects of shear which, in the lower arches of a high arch dam, are too great to be neglected. The writer has found, in dams of this type, that the effects of shear amount to as much as 12½%, or more, on computed compressive stresses, 146% on tensile stresses, and 3% on crown deflections.

Using the authors' nomenclature and Fig. 14, the movement of C about D due to shear distortion may readily be found and combined with the movements due to bending moment and thrust. Consider the voussoir, the center of which is D. Let Δr be the radial movement of its upper end relative to its lower end; V, the radial shear at D acting negatively inward; E', the modulus of elasticity of the masonry in shear; and V_L, the shear at D due to the water load between C and D. The following relations may now be written:

$$\Delta y = \sum_C^A \Delta r \cos \alpha \quad \ldots\ldots (62)$$

$$\Delta x = -\sum_C^A \Delta r \sin \alpha \quad \ldots\ldots (63)$$

$$\Delta r = \frac{n V s}{E' T} = \frac{3 V s}{E T} \quad \ldots\ldots (64)$$

$$V = H_C \sin \alpha + V_C \cos \alpha + V_L \quad \ldots\ldots (65)$$

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$$\Delta y = 3 \sum_C^{A_0} \frac{\cos \alpha}{E\,T} (H_C \sin \alpha + V_C \cos \alpha + V_L) \ldots\ldots\ldots (66)$$

$$\Delta x = -3 \sum_C^{A_0} \frac{\sin \alpha}{E\,T} (H_C \sin \alpha + V_C \cos \alpha + V_L) \ldots\ldots\ldots (67)$$

The value of Δy and Δx given in Equations (66) and (67) may now be added to Equations (22) and (23), respectively. Proceeding in a manner similar to that followed by the authors in the development of Equations (28) and (29), and noting that Δx and V_C should have their signs changed for the right half of the arch, new general equations, Equations (68) and (69), have been derived that include the combined effects of bending moment, thrust, and shear. Since shear does not affect the value of $\Delta \phi$ in Equation (10), Equation (27) remains unchanged; but Equations (28) and (29) are changed as follows:

$$M_C \left(\sum_L \frac{x\,s}{I} - \sum_R \frac{x\,s}{I} \right) + H_C \left(\sum_L \frac{x\,y\,s}{I} - \sum_R \frac{x\,y\,s}{I} + 2 \sum_L \frac{s \sin \alpha \cos \alpha}{T} - 2 \sum_R \frac{s \sin \alpha \cos \alpha}{T} \right) + V_C \left(\sum_L \frac{x^2\,s}{I} + \sum_R \frac{x^2\,s}{I} + \sum_L \frac{s \sin^2 \alpha}{T} + \sum_R \frac{s \sin^2 \alpha}{T} + 3 \sum_L \frac{s \cos^2 \alpha}{T} + 3 \sum_R \frac{s \cos^2 \alpha}{T} \right) = \sum_L \frac{M_L\,x\,s}{I} - \sum_R \frac{M_R\,x\,s}{I} + \sum_L \frac{H_L\,s \sin \alpha}{I} - \sum_R \frac{H_R\,s \sin \alpha}{T} - 3 \sum_L \frac{V_L\,s \cos \alpha}{T} + 3 \sum_R \frac{V_R\,s \cos \alpha}{T} - c\,t\,E\,(h_L - h_R) \ldots\ldots\ldots\ldots\ldots\ldots (68)$$

$$M_C \left(\sum_L \frac{y\,s}{I} + \sum_R \frac{y\,s}{I} \right) + H_C \left(\sum_L \frac{y^2\,s}{I} + \sum_R \frac{y^2\,s}{I} + \sum_L \frac{s \cos^2 \alpha}{T} + \sum_R \frac{s \cos^2 \alpha}{T} + 3 \sum_L \frac{s \sin^2 \alpha}{T} + 3 \sum_R \frac{s \sin^2 \alpha}{T} \right) + V_C \left(\sum_L \frac{x\,y\,s}{I} - \sum_R \frac{x\,y\,s}{I} + 2 \sum_L \frac{s \sin \alpha \cos \alpha}{T} - 2 \sum_R \frac{s \sin \alpha \cos \alpha}{T} \right) = \sum_L \frac{M_L\,y\,s}{I} + \sum_R \frac{M_R\,y\,s}{I} - \sum_L \frac{H_L\,s \cos \alpha}{T} - \sum_R \frac{H_R\,s \cos \alpha}{T} - 3 \sum_L \frac{V_L \sin \alpha}{T} - 3 \sum_R \frac{V_R \sin \alpha}{T} + \frac{c\,t\,E}{s} (l_L + l_R) \ldots\ldots\ldots\ldots\ldots\ldots (69)$$

M_C, H_C, and V_C, having been computed by means of Equations (27), (68), and (69), the moments, thrusts, and shears may be computed by Equations (7), (18) and (65).

If the arch is assumed to be symmetrical, Equations (27), (68), and (69) may be greatly simplified; and inasmuch as generally in the design of an arch dam it is possible and desirable to make each arch symmetrical, it will be of interest and of value to impose the conditions of symmetry on these equations. In this procedure the $\sum_L$ and $\sum_R$ for like quantities are equal

and algebraically additive. Having due regard for signs, dividing by 2 in each case, and omitting the effects of temperature, Equations (27), (68) and (69) assume the following forms:

$$M_C \sum_L \frac{s}{I} + H_C \sum_L \frac{s\,y}{I} = \sum_L \frac{M_L\,s}{I} \quad \ldots\ldots\ldots\ldots (70)$$

$$V_C = 0 \quad \ldots\ldots\ldots\ldots (71)$$

$$M_C \sum_L \frac{y\,s}{I} + H_C \left(\sum_L \frac{y^2\,s}{I} + \sum_L \frac{s \cos^2 \alpha}{T} + 3 \sum_L \frac{s \sin^2 \alpha}{T} \right) = -\sum_L \frac{M_L\,y\,s}{I} - \sum_L \frac{H_L\,s \cos \alpha}{T} - 3 \sum_L \frac{V_L \sin \alpha}{T} \quad \ldots\ldots\ldots (72)$$

M_C, V_C, and H_C, for the left half of the arch, having been found from Equations (70), (71) and (72), the moments, thrusts, and shears for the left half of the arch may be found by use of Equations (7) and (18) and Equation (65), by noting that the V_C-terms become zero. The moments, thrusts, and shears will be the same, of course, for corresponding voussoirs for the right half of the arch.

The authors have extended the customary method of distributing the computed water load on an arch dam between the horizontal arches and the single vertical cantilever through the crowns on the basis of relative rigidity. In making this extension, the cantilevers are assumed to deflect in radial planes perpendicular to the extrados of the dam. There is considerable error involved in this assumption. The sections of the horizontal arches cannot deflect in a radial direction except at the crown, and the divergence from a radial direction increases from the crown of the arch toward the abutments. All the lateral cantilevers, therefore, will be affected by the torsional force of each intersecting arch that tends to twist them around into coincidence with the motion of the arch. This tendency will be different for the horizontal arches because of their variable lengths, thicknesses, and loads. Moreover, the components of the arch-deflecting forces perpendicular to the radial plane through the central axes of the cantilevers have different intensities in the various arches, and they will tend to distort the cantilever tangentially and warp it out of shape in this direction also. The lateral cantilevers are, therefore, not only called upon to carry their share of the water load in the assumed radial directions, but must resist torsional stresses and lateral distortion stresses of considerable amounts. In this they are not dependent on their resistance alone, but on the enormous torsional and tangential resistance of the dam itself.

In order to show the directions and amounts of the deflections for various points in any single arch element and in different arch elements of the same dam, the writer has had the deflections computed for two circular symmetrical arches each of 100 ft. mean radius and 120° central angle; one with a uniform thickness of 4 ft. and a normal load of 2 500 lb. per sq. ft.; the other with a uniform thickness of 20 ft. and a normal load of 12 500 lb. per sq. ft. The results are given in Table 10 and Figs. 24 and 25 show the deflection of the 20-ft. arch ring, as an example.

It will be noted that the points, A, B, C, and D, at 15° intervals on the center line of the arches, $L\,R$ (Fig. 24), are deflected to the points, A', B',

C', and D'. To measure the deflections divide the scale values by 1 000. Curve I is the locus of points bisecting the deflections $A\,A'$, $B\,B'$, etc. The deflection of D is coincident with the assumed radial deflection of the cantilever, but the deflections at C, B, and A diverge from radial deflections by 7° 27′, 16° 59′, and 31° 27′, respectively. This lack of direction coincidence will cause torsional stress in the cantilevers. $A''\,A'$, $B''\,B'$, and $C''\,C'$, perpendicular to the respective radii, may be taken to represent graphically the peripheral deflections necessary to make the movements of the cantilevers coincide

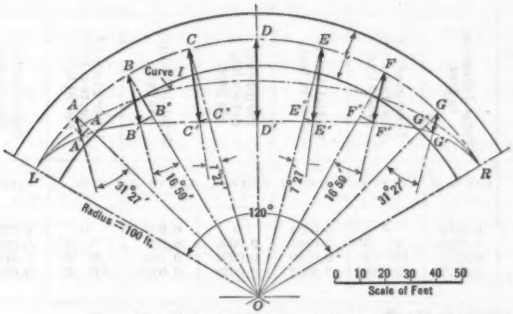

FIG. 24.—SECTION THROUGH ARCH RING.

with those of the arches. Since the points, A, B, C, and D, must deflect to the same respective points in the arches and cantilevers, the cantilevers must be distorted laterally by considerable amounts. In Fig. 25 four curves are plotted, that are of interest in connection with this subject, Curve A showing the total deflections, AA', etc.; Curve B, the radial deflections, AA'', etc.; Curve C, the peripheral deflections, $A''\,A'$, etc.; and Curve D, the divergence of the arch deflections from radial lines.

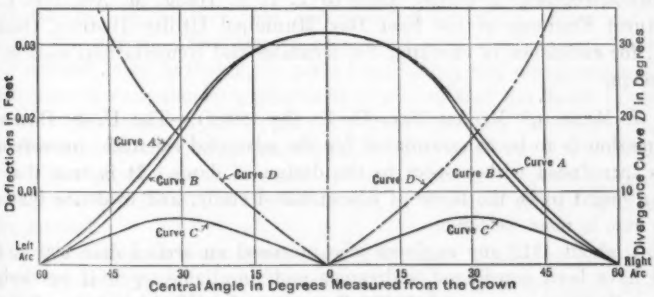

FIG. 25.—DEFLECTIONS OF ARCH RING.

In addition to the torsional and lateral distorsion forces already referred to as affecting the lateral radial cantilevers, these cantilevers are also affected by the moment, thrust, and shearing forces of the arches, gravity, and water pressure, all of which must be taken into consideration in determining their value

in the distribution of stresses in the dam. In view of these conditions, it appears that cantilevers other than the crown cantilever must be used with great care and keen judgment in order to appraise their true value in determining stress distribution in arch dams.

TABLE 10.—DEFLECTIONS OF ARCH RING UNDER UNIFORM NORMAL WATER PRESSURE.*.

Angular distance from crown, in degrees.	ARCH, 4 FEET THICK.†				ARCH, 20 FEET THICK.‡			
	Total deflection, in feet.	Divergence of deflection from radial line.	Radial deflection, in feet.	Peripheral deflection, in feet.	Total deflection, in feet.	Divergence of deflection from radial line.	Radial deflection, in feet.	Peripheral deflection, in feet.
Abscissas.*	Curve *A*.	Curve *D*	Curve *B*.	Curve *C*.	Curve *A*.	Curve *D*.	Curve *B*.	Curve *C*.
0	0.0334	0	0.0334	0	0.316	0	0.0316	0
15	0.0285	7° 5′	0.0283	0.0035	0.289	7° 27′	0.0286	0.0038
30	0.0187	16° 9′	0.0180	0.0052	0.192	16° 59′	0.0184	0.0056
45	0.0072	32° 59′	0.0060	0.0039	0.0078	31° 27′	0.0066	0.0040

* See values plotted in Fig. 25.
† $t = 4$ ft.; $w = 2\,500$ lb. per sq. ft.
‡ $t = 20$ ft.; $w = 12\,500$ lb. per sq. ft.

While the writer prefers to use the elastic center, rigid bracket, unit load method of developing arch deflection and stress formulas and a series of cantilevers of varying widths, each symmetrical with respect to a vertical section through the arch crowns of the dam, he feels that the authors have presented an excellent method of arch analysis and have rendered the Engineering Profession a distinct service for which they deserve its thanks.

Acknowledgment is hereby made to T. L. E. Haug, M. Am. Soc. C. E., Structural Engineer of the East Bay Municipal Utility District, Oakland, Calif., for assistance in checking the formulas and computations used in this discussion.

L. J. MENSCH,* M. AM. SOC. C. E. (by letter).—The U. S. Bureau of Reclamation is to be congratulated for the advanced scientific improvements it has introduced with respect to the design of dams. It is true that this Bureau ought to be the home of concentrated study, and evidence that such is the case is most welcome.

Until about 1912 any engineer who proposed an arched dam 300 ft. high would have been considered a dreamer and unreliable, even if he designed it for cylindrical stresses of 30 000 lb. per sq. ft. Nothing but a full-sized gravity dam, with an additional safeguard provided by making it an arched structure, was considered proper. Although quite a large number of successful arch dams—many of them considerably less than 100 ft. in height—had been built in the United States and foreign countries since 1870, con-

* Chicago, Ill.

servative engineers would not favor them because the distribution of stress in such structures appeared to them a mystery.

Since 1904, the Society has published quite a number of papers and discussions, in which many earnest workers have tried to clear up this subject. The general trend has been to solve the problem by assuming the arched structure to consist of two systems of members, namely, a horizontal system of arches fixed at the abutments and a vertical system of cantilevers fastened to the bed-rock.

The writer has shown, by the use of the elastic theory,* that the stresses in such horizontal arches of relatively considerable thickness are much greater than those given by the cylinder formula. He also showed that the assumed vertical cantilevers behave more like beams simply supported at the base of the dam rather than rigidly fixed at that point. Other efforts have been made to explain the arch action by the same two systems of members, giving simplified approximate formulas.† William Cain, M. Am. Soc. C. E., has treated‡ the thick cylindrical arch in a more general form.

These papers only showed that there are very much higher stresses than were formerly supposed in horizontal arches and cantilevers, but they did not explain the safety of many existing daring arched dam structures. It has been known for more than ten years that theoretical investigations along the old line were futile and the writer has previously pointed out§ that, in thick sections, secondary arches are formed which are stable and which prevent the failure of the main arch. He has also shown that arches of increased thickness at the abutments are much more economical than arches of uniform thickness.

The engineers of the Bureau of Reclamation, according to the authors, also have adopted the system of horizontal arches and vertical cantilevers, but have gone farther than the previous designers by investigating the influence of the cantilevers not only in the center of the dam, but also those nearer the abutments. In addition, they have assumed secondary arches and arches of increasing depth. They deserve great credit for this advance because it finally enabled them to design, in perfect confidence and along rational lines, arched dams of such slender proportions as no engineer has dared to consider since the construction of the Upper Otay Dam in 1900. The fact is that the dams discussed by the authors show stresses of from 80 000 to 100 000 lb. per sq. ft., if computed by the common cylinder formula (neglecting cantilever action), and from 120 000 to 190 000 lb. per sq. ft., if calculated by Professor Cain's method.

The paper shows the enormous amount of labor it takes to design such a dam by the trial method, and the slow steps by which the latest perfected design was reached. It is possible, however, to obtain results with a still

* *Transactions*, Am. Soc. C. E., Vol. LXXVIII (1915), p. 610.

† "Gravity and Arch Action in Curved Dams," by Fred A. Noetzli, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 1, and "The Relation Between Deflections and Stresses in Arch Dams," *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 284.

‡ "The Circular Arch Under Normal Loads," *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 233.

§ *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 250.

greater economy of material and labor by using more calculus and less arithmetic in the design.

Before showing this simpler method the writer wishes to restate the theory of arches under normal loads.

Let $A\,C\,B$ (Fig. 26) be the center line of a circular arch of a constant width of 1 ft. and a constant depth of t, subjected to water pressure of p lb. per sq. ft. If this arch were considered to be free at the abutments, and to act as a portion of a full circular cylinder subjected to uniform normal pressure, it would diminish in diameter and take a new position, A', C', B', concentric to its original position. The diminution of radius can be found by the following consideration: The shortening of the whole cylinder equals uniform stress per square foot, $\frac{(p\,r)}{t}$, times length of periphery of cylinder, $(2\,r\,\pi)$, divided by modulus of elasticity, (E).

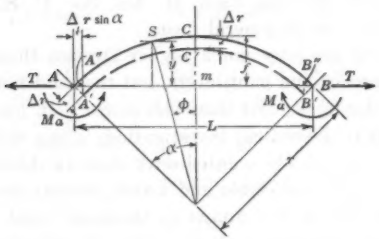

FIG. 26.

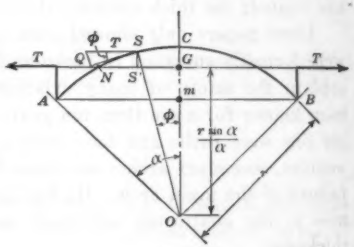

FIG. 27.

Dividing the shortening of the cylinder by $2\,\pi$ gives the shortening of the radius,

$$\Delta\,r = \frac{p\,r}{t} \times \frac{2\,r\,\pi}{E} \times \frac{1}{2\,\pi} = \frac{p\,r^2}{E\,t} \quad \ldots\ldots\ldots\ldots\ldots\ldots (73)$$

From Fig. 26 it can be seen that the chord of the unrestrained arch, A', C', B', has been shortened in each half of the span an amount, $A\,A''$, equal to $\Delta\,r \sin \alpha$, or the shortening of one-half the arch span,

$$\Delta\,x = \Delta\,r \sin \alpha = \frac{p\,r^2}{E\,t} \sin \alpha \quad \ldots\ldots\ldots\ldots\ldots\ldots (74)$$

This value is also given by Equation (14),

$$\Delta\,x = \int_0^{\alpha} \Delta\,s \cos \phi = \int_0^{\alpha} \frac{p}{E\,t}\,r\,r\,d\phi \cos \phi = \frac{p\,r^2}{E\,t} \sin \alpha \ldots\ldots (75)$$

As the arch is fixed at A and B, there have to be applied horizontal forces, T, acting outward at the abutments to lengthen the chord of the arch, $A'B'$, to the original length, $A\,B$, and moments at the abutments, M_a, to keep the ends of the arch fixed in direction.

Then the moment due to the uniform shortening of the arch at any point, S, for example, is,

$$M = T\,y - M_a \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (76)$$

Since the angles of the tangents at A and C cannot change, then according to Equation (1),

$$\Delta\theta = \int_0^a \frac{M\,d\,s}{E\,I} = 0 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (77)$$

Substituting Equation (76) in Equation (77), and transposing:

$$M_a = \frac{\int_0^a y\,d\,s}{\int_0^a d\,s} \times T \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (78)$$

The term, $\dfrac{\int_0^a y\,d\,s}{\int_0^a d\,s}$, equals $G\,m$, the distance of the center of gravity of the line, $A\,C\,B$, from the chord, $A\ B$, when G is the center of gravity of the center line of the arch, as shown in Fig. 27. It is known that $G\ O = r\,\dfrac{\sin\alpha}{\alpha}$; hence,

$$G\,m = r\,\frac{\sin\alpha}{\alpha} - r\cos\alpha = r\left(\frac{\sin\alpha}{\alpha} - \cos\alpha\right)$$

and,

$$M_a = T\,r\left(\frac{\sin\alpha}{\alpha} - \cos\alpha\right)\ldots\ldots\ldots\ldots\ldots\ldots\ldots (79)$$

Hence, substituting Equation (79) in Equation (76),

$$M = T\,(r\cos\phi - r\cos\alpha) - T\,r\left(\frac{\sin\alpha}{\alpha} - \cos\alpha\right) = T\,r\left(\cos\phi - \frac{\sin\alpha}{\alpha}\right)$$
$$= T \times S\,S' \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (80)$$

This means that it is permissible to assume T acting on a line through the center of gravity, G, of the center line of the arch (see Fig. 27).

Hence, there is only one statically unknown quantity for the solution of the effect of the shortening of the arch, namely, T, which can be found by the well known principle of least work,

$$L = \int \frac{Q^2\,d\,s}{2\left(\frac{E}{3}\right)t} + \int \frac{N^2\,d\,s}{2\,E\,t} + \int \frac{M^2\,d\,s}{E\,I} \ldots\ldots\ldots\ldots\ldots (81)$$

in which, Q, N, and M are the shear, force normal to, and moment at any section, respectively, due to the action of T only. By differentiating according to T and integrating over one-half the span only, the horizontal movement, $A''\,A$ (Fig. 26), due to the unknown force, T, can be determined as follows:

$$\frac{\delta\,L}{\delta\,T} = A\,A'' = \frac{p\,r^2}{E\,t}\sin\alpha = 3\int_0^a \frac{Q}{E\,t}\,\frac{\delta\,Q}{\delta\,T}\,d\,s + \int_0^a \frac{N}{E\,t}\,\frac{\delta\,N}{\delta\,T}\,d\,s$$
$$+ \int_0^a \frac{M}{E\,I}\,\frac{\delta\,M}{\delta\,T}\,d\,s \ldots\ldots\ldots\ldots\ldots\ldots\ldots (82)$$

From Fig. 27,

$$Q = T \sin \phi; \frac{\delta Q}{\delta T} = \sin \phi \quad \ldots\ldots\ldots\ldots\ldots\ldots (83)$$

$$N = T \cos \phi; \frac{\delta N}{\delta T} = \cos \phi \quad \ldots\ldots\ldots\ldots\ldots\ldots (84)$$

$$d s = r \, d \phi$$

From Equation (80),

$$\frac{\delta M}{\delta T} = r \left(\cos \phi - \frac{\sin \alpha}{\alpha} \right) \quad \ldots\ldots\ldots\ldots\ldots\ldots (85)$$

Substituting in Equation (82), the proper values from Equations (83), (84), and (85), and integrating:

$$\frac{p \, r^2}{E \, t} \sin \alpha = \frac{T \, r}{E \, t} \left\{ 2 \alpha - \frac{1}{2} \sin 2 \alpha \right\} + \frac{T \, r^3}{E \, I} \left\{ \frac{1}{4} \sin 2 \alpha + \frac{\alpha}{2} - \frac{\sin \alpha}{\alpha} \right\} \quad \ldots\ldots\ldots\ldots\ldots\ldots (86)^*$$

If the value, j, is substituted for $\left(2 \alpha - \frac{1}{2} \sin 2 \alpha \right)$ and the value, k, for $\left(\frac{1}{4} \sin 2 \alpha + \frac{\alpha}{2} - \frac{\sin \alpha}{\alpha} \right)$, Equation (86) becomes:

$$\frac{p \, r^2}{E \, t} \sin \alpha = \frac{T \, r}{E \, t} \left[j + \frac{12 \, r^2}{t^2} k \right] \quad \ldots\ldots\ldots\ldots\ldots\ldots (87)$$

from which, T can be readily found.

If the uniform shortening is produced by a drop of temperature, t_0, or an equivalent shrinkage of concrete, the term, $\frac{p \, r^2}{E \, t}$ (which is $\Delta \, r$) must be replaced by $c \, t_0 \, r$. This value, substituted in Equation (87), gives:

$$c \, t_0 \, r \sin \alpha = \frac{T \, r}{E \, t} \left[j + \frac{12 \, r^2}{t^2} k \right] \quad \ldots\ldots\ldots\ldots\ldots\ldots (88)$$

Equations (87) and (88) become more useful by introducing the rise of the arch, f (Fig. 26), instead of the radius:

$$f = r \, (1 - \cos \alpha)$$

For water pressure:

$$T = \frac{p \, r \sin \alpha}{j + \frac{12}{(1 - \cos \alpha)^2} \frac{f^2}{t^2} k} \quad \ldots\ldots\ldots\ldots\ldots\ldots (89)$$

For change of temperature:

$$T_0 = \frac{E \, c \, t_0 \, t \sin \alpha}{j + \frac{12}{(1 - \cos \alpha)^2} \frac{f^2}{t^2} k} \quad \ldots\ldots\ldots\ldots\ldots\ldots (90)$$

Equations (89) and (90) are more readily solved by the use of Table 11.

* *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 237, Equations (10) and (11); also, p. 249.

The equations of T in this form show the influence of the thickness of the arch very clearly. For example, assume the ratio of rise to thickness, $\frac{f}{t} = 3$, and assume $\alpha = \frac{\pi}{6}$. Then, from Table 11,

$$T = \frac{p\ r}{1.13 \times 9 + 1.228} = \frac{p\ r}{11.398} = 0.0878\ p\ r.$$

T acts at a point, below the crown, $r - r\ \frac{\sin\ \alpha}{\alpha} = r - 0.95494\ r = 0.0451\ r$. The rise of the arch, f, $= 0.133\ r$, and $t = \frac{f}{3} = 0.0444\ r$. Hence, the shortening of the arch in this case produces a moment at the crown equal to $p\ r \times 0.0878 \times 0.0451\ r = 0.00396\ p\ r^2$; a moment at the abutment of $0.0878\ p\ r\ (0.133\ r - 0.0451\ r) = 0.00775\ p\ r^2$; a normal force at the crown of $p\ r - 0.0878\ p\ r = 0.9122\ p\ r$; and a normal force at the abutment of $p\ r - 0.0878\ p\ r \cos 30 = 0.924\ p\ r$.

TABLE 11.—TABLE OF CONSTANTS FOR VARIOUS VALUES OF α.

α.	j.	k.	$\frac{\sin\ \alpha}{\alpha}$.	f.	T.	T_0.
$\frac{\pi}{6}$	0.642	0.00084	0.9549	$0.133\ r$	$\frac{p\ r}{1.13\ \frac{f^2}{t^2} + 1.228}$	$\frac{E\ c\ t_0\ t}{1.13\ \frac{f^2}{t^2} + 1.228}$
$\frac{\pi}{4}$	1.0708	0.00608	0.900	$0.293\ r$	$\frac{p\ r}{1.21\ \frac{f^2}{t^2} + 1.515}$	$\frac{E\ c\ t_0\ t}{1.21\ \frac{f^2}{t^2} + 1.515}$
$\frac{\pi}{3}$	1.6614	0.0239	0.8276	$0.50\ r$	$\frac{p\ r}{1.33\ \frac{f^2}{t^2} + 1.92}$	$\frac{E\ c\ t_0\ t}{1.33\ \frac{f^2}{t^2} + 1.92}$
$\frac{\pi}{2}$	3.1416	0.1488	0.6366	$1.00\ r$	$\frac{p\ r}{1.785\ \frac{f^2}{t^2} + 3.14}$	$\frac{E\ c\ t_0\ t}{1.785\ \frac{f^2}{t^2} + 3.14}$

The eccentricity of the normal force at the crown is,

$$\frac{0.00396\ p\ r^2}{0.9122\ p\ r} = 0.00435\ r = 0.098\ t$$

and that at the abutment is,

$$\frac{0.00775\ p\ r^2}{0.924\ p\ r} = 0.0084\ r = 0.189\ t$$

While the eccentricity at the crown is less than one-tenth the thickness of the arch, the point of application of the normal force at the abutment is outside the middle third. From the equations in Table 11, it is clear that tensile stresses are produced in all arches, whether flat or steep, whenever $\frac{f}{t}$ is less than 3.

The tensile stresses are, in fact, much larger than shown when a fall of temperature or shrinkage of concrete is taken into consideration, as the following calculations show.

Assume in the same arch, $p = 4\,000$ lb.; $r = 270$ ft.; $f = 270\ (1 - \cos 30) = 36$ ft.; $t = \frac{f}{3} = 12$ ft.; then,

$$E\ c\ t_0 = 288 \times 10^6 \times 0.0000055 \times 12 = 19\,000 \text{ lb.}$$

for the very low drop of only 12°, and from Table 11,

$$T_0 = \frac{E\ c\ t_0\ t}{1.13\ \frac{f^2}{t^2} + 1.228} = \frac{19\,000 \times 12}{11.398} = 20\,000 \text{ lb.}$$

The moment at the crown is,

$$20\,000 \times \left(r - r\,\frac{\sin\,\alpha}{\alpha}\right) = 20\,000 \times 0.0451\ r = 243\,000 \text{ ft-lb.}$$

and the moment at the abutment is,

$$20\,000 \times (f - 0.0451\ r) = 20\,000 \times 0.0882\ r = 478\,000 \text{ ft-lb.}$$

The normal force at the crown is,

$$0.9122\ p\ r - 20\,000 = 965\,000 \text{ lb.}$$

and the normal force at the abutment is,

$$0.924\ p\ r - 20\,000 \cos 30 = 979\,000 \text{ lb.}$$

The eccentricity at the crown due to temperature only is,

$$\frac{243\,000}{965\,000} = 0.252 \text{ ft.} = 0.021\ t$$

and the eccentricity at the abutment due to temperature only is,

$$\frac{478\,000}{979\,000} = 0.489 \text{ ft.} = 0.0407\ t$$

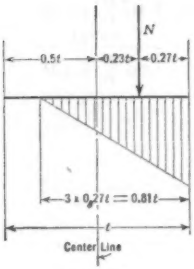

FIG. 28.

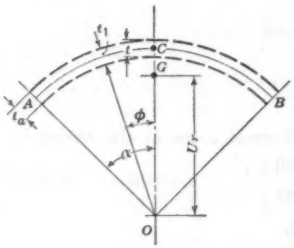

FIG. 29.

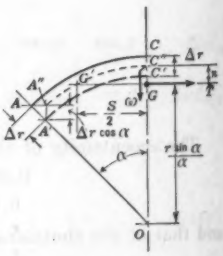

FIG. 30.

Hence, the total eccentricity from shortening of the arch and drop of temperature at the crown $= 0.098\ t + 0.021\ t = 0.119\ t$, and at the abutment $= 0.189\ t + 0.0407\ t = 0.230\ t$.

The maximum stress (see Fig. 28) $= \frac{N}{0.81\ t} \times 2 = 2.48\ \frac{N}{t}$, or nearly $2\frac{1}{2}$ times the stress from the cylinder theory.

Therefore, where $\frac{f}{t}$ is less than 3, secondary arches are forming and economical design requires that the arch should not be of constant thickness throughout but should have its section enlarged where tensile stresses occur.

If the influence of shear is omitted from consideration,*

$$T = \frac{p\,r}{1.130\,\frac{f^2}{t^2} + 0.9568} = 0.0899\,p\,r$$

instead of $0.0878\,p\,r$; or T is 2% larger than when shear is considered.

Where (as in the Gibson Dam) $\frac{f}{t} = 1$, it is easily found that T, without consideration of shear, becomes 7% larger; but where (as in the lower part of the Bear Valley Dam), $\frac{f}{t} = 0.22$, Table 11 shows that,

$$T = \frac{p\,r}{1.13 \times 0.22^2 + 1.228} = 0.778\,p\,r$$

and from Professor Cain's formula,

$$T = \frac{p\,r}{1.13\,\frac{f^2}{t^2} + 0.9568} = 0.989\,p\,r$$

This is a great difference and it shows that the dam could not exist without secondary arch action.

None of these formulas applies to non-reinforced arches in which tensile stresses occur, and such stresses do occur in nearly all arches (plain and reinforced), in which $\frac{f}{t}$ is less than 3. Secondary arches are then acting within such arches, having a larger rise, a smaller radius, a smaller thickness, and also a different pressure than the original arch. The water pressure will be different because the water will intrude in the cracks at the extrados near the abutments.

It is known that the eccentricity of the normal force at the abutments is about twice as large as that at the crown, and it is a mystery to the writer why arched dams are not designed in the first place in the shape of the future secondary arches, with enlargements at the dangerous sections.

Possibly one reason is that the summation method of designing arches of various depths is too laborious for most engineers. Secondary arches, such as have been used by the Bureau of Reclamation, can be much more simply designed by the following method.

Let $A\,C\,B$ (Fig. 29) be the center line of a circular arch having a thickness of t at the crown and t_a at the abutment. Assume that the thickness varies as:

$$\frac{1}{t_1} = \frac{1}{t}\left(1 - K\,\frac{\sin\phi}{\sin\alpha}\right) \qquad (91)$$

* *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 233.

in which, K is a constant; I = the moment of inertia at the crown; I_a = the moment of inertia at the abutment; and I_1 = the moment of inertia at any point.

For all practical purposes:

$$\frac{1}{I_1} = \frac{1}{I}\left(1 - L\frac{\sin\phi}{\sin\alpha}\right) \quad \ldots\ldots (92)$$

in which, L is another constant. When the thickness of the abutment is $1\frac{1}{2}$ times the thickness at the crown, K may be assumed as $\frac{1}{3}$ and the corresponding L as $\frac{3}{4}$. When the thickness at the abutment is 2, assume that $K = \frac{1}{2}$ and the corresponding $L = \frac{9}{10}$. The writer has checked this assumption in numerous cases with the summation method and found only very slight variation in results.

In order to solve the arch (Fig. 29) by the same method as for an arch of constant depth, it is necessary to know the new value, $A\,A'' = \Delta\,x$ (see Fig. 30) and the new line in which T is acting, which does not now go through the center of gravity of the arch but through the center of gravity of the so-called elastic weight, $\frac{d\,s}{I_1}$. By the use of Equation (75) modified by Equation (91),

$$\Delta\,x = \int_0^a \Delta\,s\cos\phi = \int_0^a \frac{p\,r}{E\,t}\left(1 - K\frac{\sin\phi}{\sin\alpha}\right) r\,d\,\phi\cos\phi$$

$$= \frac{p\,r^2}{E\,t}\sin\alpha\left(1 - \frac{K}{2}\right) \quad \ldots\ldots (93)$$

The new distance, $G\,O$ (Fig. 30), is,

$$\frac{\int_0^a \frac{y\,d\,s}{I_1}}{\int_0^a \frac{d\,s}{I_1}} = \frac{\frac{r^2}{I}\sin\alpha\left(1 - \frac{L}{2}\right)}{\frac{r}{I}\left[\alpha - L\frac{(1-\cos\alpha)}{\sin\alpha}\right]} = \frac{\left(1 - \frac{L}{2}\right) r\sin}{\alpha - L\frac{(1-\cos\alpha)}{\sin\alpha}} = U\,r. \quad (94)$$

Assigning actual values to α, the corresponding values of U (See Equation (94)) for values of $t_a = 1\frac{1}{2}\,t$, and $2\,t$, respectively, are as given in Table 12.

The moment at any point, due to the shortening of the arch, is:

$$M = T\,(r\cos\phi - U\,r) \quad \ldots\ldots (95)$$

In the least work equation,

$$L = \int \frac{Q^2\,d\,s}{2\left(\frac{E}{3}\right)t_1} + \int \frac{N^2\,d\,s}{2\,E\,t_1} + \int \frac{M^2\,d\,s}{E\,I_1} \quad \ldots\ldots (96)$$

Let,

$$Q = T\sin\phi\,;\ \frac{\delta\,Q}{\delta\,T} = \sin\phi \quad \ldots\ldots (97)$$

$$N = T\cos\phi\,;\ \frac{\delta\,N}{\delta\,T} = \cos\phi \quad \ldots\ldots (98)$$

$$M = T\,r\,(\cos\phi - U);\quad \frac{\delta M}{\delta T} = r\,(\cos\phi - U)\ldots\ldots\ldots\ldots(99)$$

$$\frac{d\,s}{t_1} = \frac{r\,d\,\phi}{t}\left(1 - K\frac{\sin\phi}{\sin\alpha}\right);\ \text{and}\ \frac{d\,s}{I_1} = \frac{r\,d\,\phi}{I}\,1 - L\frac{\sin\phi}{\sin\alpha}\ldots\ldots(100)$$

TABLE 12.—Tables of Constants for Corresponding Values of α.

α.	$t_a = 1\frac{1}{2}\,t$.				$t_a = 2\,t$.			
	U.	j.	k.	T.	U.	j.	k.	T.
$\frac{\pi}{6}$	0.968	0.5020	0.00038	$\frac{pr}{0.64\,\frac{f^2}{t^2} + 1.20}$	0.970	0.4459	0.000268	$\frac{pr}{0.48\,\frac{f^2}{t^2} + 1.19}$
$\frac{\pi}{4}$	0.931	0.8619	0.002733	$\frac{pr}{0.65\,\frac{f^2}{t^2} + 1.46}$	0.943	0.7542	0.00186	$\frac{pr}{0.491\,\frac{f^2}{t^2} + 1.40}$
$\frac{\pi}{3}$	0.880	1.3086	0.010611	$\frac{pr}{0.707\,\frac{f^2}{t^2} + 1.81}$	0.903	1.1322	0.006968	$\frac{pr}{0.515\,\frac{f^2}{t^2} + 1.74}$
$\frac{\pi}{2}$	0.762	.....	.....	.....	0.819	.....	.....	.....

Substituting Equation (93) in Equation (96) and differentiating according to T, $\frac{\delta L}{\delta T} = \Delta x$, is the movement of one abutment. Then,

$$\Delta x = \frac{p\,r^2}{E\,t}\sin\alpha\left(1 - \frac{K}{2}\right) = \frac{3\,T\,r}{E\,t}\int_0^\alpha \sin^2\phi\left(1 - K\frac{\sin\phi}{\sin\alpha}\right)d\,\phi$$
$$+ \frac{T\,r}{E\,t}\int_0^\alpha \cos^2\phi\left(1 - K\frac{\sin\phi}{\sin\alpha}\right)d\,\phi + \frac{T\,r^3}{E\,I}\int_0^\alpha (\cos\phi - U)^2$$
$$\left(1 - L\frac{\sin\phi}{\sin\alpha}\right)d\,\phi\ldots\ldots\ldots\ldots\ldots\ldots\ldots(101)$$

This reduces again to the form:

$$\frac{p\,r^2}{E\,t}\sin\alpha\left(1 - \frac{K}{2}\right) = \frac{T\,r}{E\,t}\left\{j + \frac{12\,r^2}{t^2}k\right\}\ldots\ldots\ldots(102)$$

The values of j, k, and T for various values of α, are as given in Table 12. The change of temperature, T_0, is obtained by substituting the expression, $\Delta x = c\,t_0\,r\sin\alpha$, in Equation (96). Then, for values of $\alpha = \frac{\pi}{6}$ and $t_a = 1\frac{1}{2}\,t$,

$$T_0 = \frac{E\,c\,t_0\,t}{\left(1 - \frac{K}{2}\right)0.64\,\frac{f^2}{t^2} + 1.20}\ldots\ldots\ldots\ldots(103)$$

A study of Table 12 shows that the expression for T will hardly change for values of α smaller than $\frac{\pi}{6}$, and it will be quite as permissible to use the

same formula for smaller values of α as for $\frac{\pi}{6}$. The authors advise using Fig. 4 for estimating the change of length, due to a drop of temperature. Presumably this includes the influence of shrinkage also. The Bureau of Reclamation has probably the most complete information on this subject, and a statement issuing from its office, containing observations on which this diagram is based, would be highly welcome. What influence has the gradual widening of the arch to $1\frac{1}{2}\ t$, near the abutment, on the stresses of an arch as discussed on pages 1255 and 1256?

From Table 12:

$$T = \frac{p\ r}{0.64 \times 3^2 + 1.20} = 0.144\ p\ r$$

From Equation (103),

$$T_0 = \frac{19\,000 \times 12}{\left\{\left(1 - \frac{1}{6}\right) 0.64 \times 3^2\right\} + 1.20} = 39\,400 \text{ lb.}$$

N at the crown equals,

$$p\ r - 0.144\ p\ r - 39\,400 = 0.856\ p\ r - 39\,400 \text{ lb.} = 885\,000 \text{ lb.}$$

N at the abutment equals,

$$p\ r - (0.144\ p\ r + 39\,400) \cos 30 = 0.8772\ p\ r - 34\,000 \text{ lb.} = 913\,000 \text{ lb.}$$

M at the crown equals,

$$(T + T_0)\ (1 - U)\ r = 195\,000 \times 0.032\ r = 1\,670\,000 \text{ ft-lb.}$$

M at the abutment equals,

$$(T + T_0)\ (U\ r - r \cos 30) = 195\,000 \times 0.101\ r = 5\,350\,000 \text{ ft-lb.}$$

Eccentricity at the crown equals,

$$\frac{1\ 670\ 000}{885\ 000} = 1.89 \text{ ft.} = 0.1575\ t$$

Eccentricity at the abutment equals,

$$\frac{5\ 350\ 000}{913\ 000} = 5.86 \text{ ft.} = 0.325\ t_a$$

The maximum stress at the abutment (see Fig. 28) is,

$$S = \frac{N}{3\left(\frac{1}{2}\ t_a - 0.325\ t_a\right)} \times 2 = 3.82\ \frac{N}{t_a} \quad \ldots\ldots\ldots\ldots (104)$$

or, notwithstanding the enlargement of the arch, the stresses, due to the shortening of the arch and a drop of temperature, become larger than before, which shows that the selection of a shape of the arch, according to Fig. 29, is not always a fortunate one.

In the authors' method of designing it is very important to find the deflection of the arches at several points. This can be done simply by the following consideration: In Fig. 30, assume $A\ C$ to be the center line of the arch and, after uniform compression, the arch moved upward into the position, $A''\ C''$. Then apply a force, T, in order to bring A'' back to A.

In this position the crown of the arch, C'', is already lowered a distance,

$$\Delta r - \Delta r \cos \alpha = \frac{p r^2}{E t}(1 - \cos \alpha) \ldots\ldots\ldots\ldots\ldots (105)$$

The deflection can be found* by assuming the left half of the arch to be fixed at A'', and the force, T, acting at G and a fictive force, w, acting at the crown, then by differentiating Equation (81) according to w, making $w = 0$ afterward:

$$Q = T \sin \phi + w \cos \phi; \frac{\delta Q}{\delta w} = \cos \phi \ldots\ldots\ldots\ldots\ldots (106)$$

$$N = - T \cos \phi + w \sin \phi; \frac{\delta N}{\delta w} = \sin \phi \ldots\ldots\ldots\ldots\ldots (107)$$

$$M = - T r \left(\cos \phi - \frac{\sin \alpha}{\alpha}\right) + w r \sin \phi; \frac{\delta M}{\delta w} = r \sin \phi \ldots\ldots (108)$$

The vertical movement of the crown of the arch in position, $A'' C'' B''$ (Fig. 30) is:

$$\frac{\delta L}{\delta w} = 3 \int_0^\alpha \frac{T r}{E t} \sin \phi \cos \phi \, d \phi - \int_0^\alpha \frac{T r}{E t} \sin \phi \cos \phi \, d \phi$$

$$- \int_0^\alpha \frac{T r^3}{E I}\left(\cos \phi - \frac{\sin \alpha}{\alpha}\right) \sin \phi \, d \phi \ldots\ldots\ldots\ldots\ldots (109)$$

Integrating and adding $\Delta r (1 - \cos \alpha)$, the total deflection of the crown is,

$$\Delta f = \frac{p r^2}{E t}(1 - \cos \alpha)$$

$$+ \frac{T r}{E t}\left\{ \sin^2 \alpha + \frac{12 r^2}{t^2}\left[\frac{\sin \alpha}{\alpha}(1 - \cos \alpha) - \frac{\sin^2 \alpha}{2}\right]\right\} \ldots\ldots (110)$$

The vertical displacement of an arch with a depth increasing from the crown to the abutment is found, similarly, by using Equations (91) and (92) in Equation (81).

The vertical displacement of the point, G', in relation to C'' is easily found from the well known equation,

$$\Delta y = \int \frac{M x \, d s}{E I} \ldots\ldots\ldots\ldots\ldots\ldots (111)$$

Consider $G' C''$ as y in the well known equation,

$$y = \frac{4 n}{S^2}(S x - x^2) \ldots\ldots\ldots\ldots\ldots\ldots (112)$$

with the origin at G': $M = T y$ for this section, and, with normal force and shear neglected,

$$\Delta y = \int_0^{\frac{s}{2}} - \frac{T y x \, d s}{E I} = \frac{4 n}{S^2} \frac{1}{E I} \int_0^{\frac{s}{2}} (S x^2 - x^3) \, d x = \frac{5}{48} \frac{T s n}{E I} \ldots\ldots (112)$$

This point and the tangents at the crown and abutment being known, it should not be much trouble to draw the deflection for the whole arch.

The writer has not mentioned Poisson's ratio because most engineers find it safer to forget it. Positive proof of its beneficial influence is still sadly lacking. The so-called curved beam action has been ignored, because theory

* *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 236.

and tests show that, for the extraordinary case where $\frac{r}{t} = 1$, the stresses are influenced 25%; for the very rare case of $\frac{r}{t} = 4$, they are influenced 9%, and for $\frac{r}{t} = 10$ (still a very thick arch), they are influenced to the amount of 4 per cent.

The stresses in the lower arches of a dam are very high, due to the shortening of the arch and a drop of temperature and shrinkage. Very daring dams are standing, however, and an explanation is urgently needed.

In seeking help from the cantilevers near the crown of the arch, it is found that the cantilevers themselves, according to theory, are stressed to the limit in nearly all cases.

The authors sought help from the cantilevers near the abutments. They assumed that the cantilevers at the crown take a very much smaller load than was formerly ascribed to them by most engineers and, in fact, they considered the horizontal arches to have a horizontal and vertical abutment at the end. This amounts, in effect, to the assumption of a horizontal arch of a smaller span.

In large arch dams as a rule, it is uneconomical to assume the arch rings to be affected only by water pressure. If the weight of the arch ring is also considered, it will be found that the most economical design is obtained by inclined arches, the inclination being given by the water pressure per square foot to the weight of the inclined ring per linear foot.* The importance of this statement will be seen in the following demonstration.

Fig. 31 shows a section through the crown of the Dam No. II, mentioned by the authors. At Elevation 5 280, the water pressure is 10 400 lb. per sq. ft., the weight of the arch ring is 5 200 lb. per lin. ft., and the ratio of water pressure and weight of ring is exactly 2 to 1.

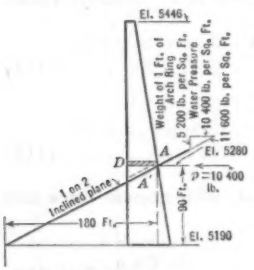

FIG. 31.

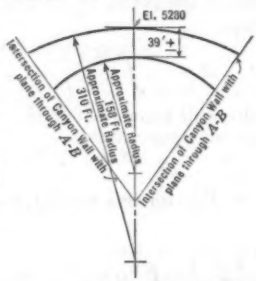

FIG. 32.

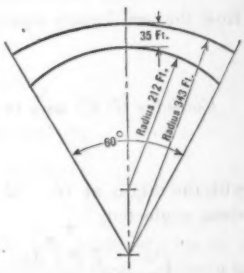

FIG. 33.

Draw Line $A\,A'$ with an inclination of 1 on 2, and lay, through Line $A\,A'$, a plane perpendicular to the plane of the paper. This plane will intersect the dam as shown in Fig. 32, while the horizontal section $A\,D$, is shown in Fig. 33. It is apparent that the radius of the inclined arch is very much shorter than the horizontal arch and will, by the principle of least work, support a much

* *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 253.

greater part of the water pressure than the horizontal arch. Of course, the inclined arch has to carry a larger load, the resultant of both vertical and horizontal load, namely, 11 600 lb. per lin. ft. Its thickness is also about 10% less than that of the horizontal arch, but it is 10% deeper and the transmission of the water pressure to the assumed arch will be different from that shown in Fig. 31. The water pressure will center around A' and will be transmitted by shear and friction. The assertion that $A\,A'$ ought to be inclined in exactly this ratio should be modified to admit that there might be cases, depending on the configuration of the canyon, where another inclination will be of greater advantage.

The writer still maintains that it would be of much greater satisfaction to most engineers if cantilever action were entirely ignored with respect to the assistance cantilevers would give to horizontal or inclined arches, and he recommends starting the design of an arched dam by properly designing the inclined arches, and assuming them to carry their own weight and water pressure, these inclined arches to be affected by a shortening of the arch and a drop of temperature.

The writer is quite convinced that the arches are, in fact, very much stronger than given by the standard design for the following reasons.

It is well known that a concrete beam, when tested to destruction, is much stronger than given by the formula,

$$M = S\,\frac{b\,d^2}{6} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (114)$$

In fact, S, computed from bending tests, is generally twice as high as the estimated tensile strength of the concrete. The reason is that the stresses do not follow Hooke's law; and instead of a straight line distribution of stresses, there is, especially near the ultimate load, a parabolic distribution of some power greater than 2.

Tests on eccentrically loaded columns also show a much greater strength, than the old straight line formula indicates.*

Assuming a distribution of stress based on a cubic parabola (see Fig. 34), tensile stresses are found to act in a rectangular section, when the eccentricity of loading is larger than $\frac{1}{10}\,t$ $\left(\text{not } \frac{1}{6}\,t \text{ as according to the old theory}\right)$; but the maximum stresses are then only $1.33\,\frac{N}{t}$.

Considère† compared a 16-in. column, loaded with an eccentricity of 4 in., with an identical column axially loaded. The latter failed at 3 600 lb. per sq. in., while the former failed (according to the old theory) at 4 950 lb. per sq. in. As the concrete strength was only 3 600 lb. per sq. in., the column actually carried $\frac{4\,950}{3\,600} = 1.37$ times the load the old theory would lead one to expect, and this result agrees, within 1%, with the cubic parabola stress dis-

* *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919-20), p. 1671.

† Rept., Commission du Beton Armé, 1907.

tribution. Careful comparisons with many other tests* also check the writer's theory.

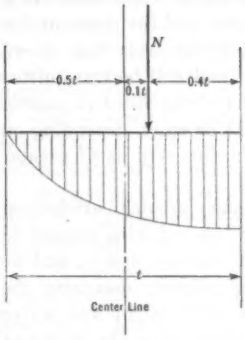

FIG. 34.

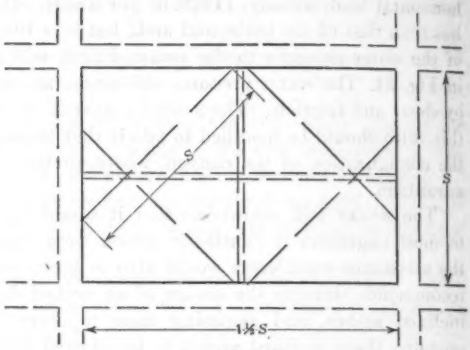

FIG. 35.

Table 13 lists the maximum stresses for various concrete sections having no tensile strength (on account of joints), e being the eccentricity and t, the depth of a rectangular section, 1 ft. wide.

TABLE 13.—MAXIMUM STRESSES IN COLUMNS FOR VARIOUS VALUES OF e.

$\frac{e}{t}$.	$\frac{N}{t}$ according to cubic parabola stress distribution.	$\frac{N}{t}$ according to straight line formula.
0.017	1.06	1.104
0.080	1.10	1.18
0.051	1.17	1.306
0.084	1.28	1.504
0.100	1.33	1.60
0.120	1.41	1.72
0.140	1.48	1.84
0.160	1.57	1.96
0.180	1.67	2.08
0.200	1.78	2.22
0.260	2.22	2.78
0.340	3.33	4.16
0.420	6.67	8.35
0.480	20.00	33.33

Table 13 shows that the stresses are in most cases 20% smaller than was formerly believed. The arches are, besides, much stronger than the common elastic theory indicates for the reason that the modulus of elasticity does not remain constant. At the abutments, the stresses first reach high values, and the modulus of elasticity becomes rapidly smaller. This means that the deformation of the arch near the abutments is relatively larger than the theory assumes. It has the effect of lowering the point of application of T, so that the greatest moment (which occurs always at the abutments), is decreased and the moment at the crown is increased. A similar effect is found in contin-

* *Beton und Eisen*, 1916, p. 56; and, also, *Forschungsarbeiten*, Verein Deutscher Ingenieure, Heft 166–169, 1914.

uous beams, and this induced the Joint Committee on Specifications for Concrete and Reinforced Concrete to recommend that continuous beams be fully loaded and fully fixed at the support in order to be designed for a moment of $\frac{p\,l^2}{16}$ $\left(\text{instead of } \frac{p\,l^2}{12} \text{ at the support, and } \frac{p\,l^2}{24} \text{ in the center}\right)$.

It is very probable that for this reason alone the arches are 33% stronger than the elastic theory teaches. It is high time that the colleges discard the archaic hypotheses, on which they build their theories. Tests ought to be made with the proper spirit of finding the true laws of the strength of materials, regardless of tradition, and tests to destruction on small model dams would soon demonstrate to the most conservative engineers the enormous strength of arched dams.

The problem of the arched dam has a corollary in the slab, supported on four beams used in building construction. In Fig. 35 such a slab is shown with a center strip in two directions. In all building ordinances in Germany, it is required that the division of the load for each system of strips throughout the center slab shall be assumed in the inverse ratio of the fourth power of their respective spans, for the only reason that the center strips have the same deflection at the center of the slab, when uniformly loaded in that ratio. Even in the United States building ordinances require a division according to the third power of the ratio of the spans. In France, however, it was found by tests, nearly thirty years ago, that the division of the loads ought to be according to the second power of the ratio of the spans. The explanation was not forthcoming for a long time, until an Italian engineer (Danusso) demonstrated that diagonal strips, near the corners, as shown in Fig. 34, greatly reduce the span in the long direction. The authors have demonstrated a very similar fact for the arched dam.

William Cain,* M. Am. Soc. C. E. (by letter).—The writer agrees with the authors that, for the complete solution of the arch dam, the whole dam must be considered, which thus involves the shape of the canyon. Friction between the horizontal arches and between the vertical cantilevers is ignored and a series of proportional loads on these two elements are, in turn, assumed until one series is found to give the same radial deflections at the same points for both elements. Of course, for an exact solution, the friction referred to should be considered as it plays a very important rôle in transferring stress toward the abutments, particularly across cracks (either vertical or horizontal) and it doubtless increases in going from the crown to the abutments where the deflection is zero. This friction or shear is not of much importance in a dam, without cracks, that is fixed (encastrée) at the base; but for the usual arch dam, that is not fixed at the base, and, with cracks, it adds materially to the strength and stability of the structure. Necessarily, by the authors' tentative method of solution it has to be ignored.

With regard to the division of the normal water pressure between a horizontal arch and the vertical cantilevers crossing it at the depth considered, some general conclusions can be given. To avoid going over old ground, the

* Prof. Emeritus, Univ. of North Carolina, Chapel Hill, N. C.

writer would refer to his discussion of the paper by Fred A. Noetzli, M. Am. Soc. C. E., entitled, "Gravity and Arch Action in Curved Dams",* especially to Fig. 15 in connection with the analysis for the Wooling Dam. The diagram refers to a cantilever at the crown, but it equally applies to one anywhere if subjected to its proper proportional water load, the arch deflection being computed for the point considered. On any cantilever, there is one point, K, where all the water pressure is carried by the arch and none by the cantilever. Under its proportional water load, the cantilever would bend so as to be concave down stream; but above K, it is found that, under this load, it tends to deflect more than the arch. This excess deflection is resisted by the arch, which exerts a horizontal pull on it acting up stream with a consequent equal reaction from the cantilever. The part of the cantilever above K thus bends so as to become concave up stream. The smaller the down-stream deflection of the arch, the greater its pull on the cantilever; so that for points above K, if the deflection of the arch decreases in going from the crown, C, to the abutment, A, the up-stream pulls on the cantilever should increase from C to A, provided all the cantilevers are of the same stiffness, as for a level dam site with cantilevers of the same size and height, that are not cracked or that have similar cracks.

The same conclusions hold when the dam site is a canyon, since then the stiffness of the cantilevers increases in going from C to A. If the deflection of the arch decreases irregularly in going from C to A, or if the cracks in the cantilever are not similar, then the variation in the load on the cantilever will likewise be irregular.

Next, consider points below K on any cantilever on which the total horizontal water pressure is p lb. per sq. ft.

Let p' = the unit normal pressure carried by the arch; and,
p'' = the unit normal pressure carried by the cantilever; so that
$p' + p'' = p$, a constant.

First, assume that, for a level dam site, the radial deflections of the arch diminish from the crown, C, to the abutment, A, where the deflection is zero. Then, since the deflections of the arch and any cantilever, at the same point, must be equal, it follows that the deflections of the cantilevers must decrease from a maximum at the crown to zero at the abutment. Further, suppose the cantilevers to be alike and without cracks or with exactly similar cracks similarly placed, so that each one will deflect the same amount for the same load; then, in order that the deflection shall decrease from some value at C, to zero at A, the unit normal pressure p'', acting on a cantilever must decrease in going from C to A. Consequently, since $p' + p'' = p$ (a constant), then p', the unit normal pressure on the arch, must increase in going from C to A.

If, however, it is assumed that the more central cantilevers are the only ones weakened by cracks, then p'' can be much less than before at C, and it may actually increase from C to a point near the abutment. The fixing of the cantilever by the side walls at the abutment will prevent any deflection, no matter what the value of p'', and this influence will extend some distance out,

* *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), pp. 73–79.

so that it is possible for p'' (and therefore, p') to remain constant from C to A, or for p'' to increase, in which case, p' decreases in going from C to A.

In the case of the canyon, Fig. 1 (c), the cantilevers becoming shorter in going from C to A, increase in stiffness and their deflections decrease for the same load, so that it is possible for p'' to remain constant throughout (considering the fixation caused by the side walls). In that case, p' is constant, or p'' can actually increase and p' can decrease in going from C to A, particularly if cracks occur in some of the cantilevers in the middle portion. In no case is it necessary that p' should be zero at A, as seems to be assumed by the authors.

All the deflections were assumed to be down stream and to decrease in going from C to A. In the arches at the crest, near the crown of the Stevenson Creek Experimental Dam, the deflection was down stream at the crown, but up stream nearer the abutment, the neutral line forming a reversed curve. Evidently, the loading on the arch was such as to produce this curve and thus to prove an exception to what would naturally be expected. From all that precedes, it is seen that the proportionate loads sustained by arch and cantilever cannot be predicted. Such loads may increase or decrease in going from crown to abutment, or they may be nearly uniform, depending on the shape of the canyon and the cracks, or portions subjected to tension, particularly in the cantilevers.

In Fig. 5, referring to Dam No. I, the load over most of the horizontal arch was uniform. In this dam, tension was supposed to be provided for; but in the other dams, where the material everywhere was supposed to be incapable of taking tension, the proportional loadings were very variable. Most arch dams are not fixed (encastrée) at the base in that no provision is made for the inevitable tension at that point for dams supposed to be fixed at the base. The authors have properly excluded the parts of the sections where tension is found, which occurs when the resultant on the section falls outside its middle third, so that only the part of a section in compression is considered in the computation.

The cantilever loads must include the weight of the concrete above the section and that of the water vertically over the up-stream face, the negative load due to the arches near the top, and the assumed proportional part of the water load on the part of the cantilever above the section considered. When the resultant of such loads cuts the section outside its middle third, say, at a distance, c, from the nearest edge, then only a length, $3c$, of the section, measured from the edge, will be supposed to be in compression; also, the neutral axis will be taken at the center of this compressed portion, and the moments and moment of inertia must be taken about this center. For sections where the resultant falls within the middle third, the moments are taken, as usual, about the center of the section.

After these moments at the various sections have been computed, the deflections of the cantilever at various points are ascertained. This can be effected by calculation, but the writer prefers a neat graphical construction,* since the deflections can be read off to 0.001 in. with practical accuracy.

* *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 74.

The authors do not make the usual erroneous assumption that the cantilever is fixed at the base. It is simply supposed to be supported at the base and to have there a vertical tangent to the neutral line. It can only be supposed to be fixed at the base when sufficient tension can be exerted, and this is ignored in the computation.

For very thick dams, the deflection due to shear should be added to that corresponding to moments. This is easily computed from the well-known formula for the deflection in feet, h ft., above the lower section, Fig. 36,

$$\Delta = \frac{1.2}{144\ G} \int_0^h \frac{S}{a}\, d\,x$$

corresponding to a parabolic distribution of shear on a cross-section of area, a. In this formula,

S = total horizontal shear due to loads acting on the cantilever, x ft. above the lower section;

a = area of horizontal section, the shear of which is S;

G = modulus of rigidity, in pounds per square inch.*

The integral gives the value of the shaded area, so that the deflection, in inches, is, for a trapezoidal section,

$$\Delta = \frac{0.1}{G} \frac{1}{2} \left(\frac{S_1}{a_1} + \frac{S_2}{a_2} \right) h$$

If preferred, the shaded area can be written, $\frac{S_0}{a_0} h$, in which, S_0 and a_0 represent the shear and area, respectively, of the section of the cantilever at the distance, $\frac{h}{2}$, above the lower section. The cantilever having been divided into a number of parts, each of approximate trapezoidal section, the deflection of the top of each part with respect to its base can be computed from this formula and the sum taken for the total deflection at a given height.

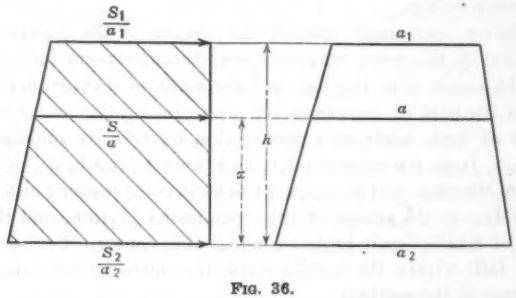

FIG. 36.

It will be seen from this discussion that the authors' method, of counting as effective only the parts of the sections of cantilevers in compression in computing moments, must necessarily lead to different results from those found

* In *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 100, B. F. Jakobsen, M. Am. Soc. C. E., gives reasons for taking $G = \frac{E}{2.4}$

on the assumption that the cantilever can resist the tension. As a matter of fact, American dams are not usually reinforced (the Wooling Dam is the only reinforced dam as far as the writer knows) and cracks are to be expected. Where they occur, it is manifestly absurd to count on tension being exerted across the cracks, or that the dam remains intact, and yet this assumption is the basis of the usual analysis. This basis, not agreeing with the facts, should be rejected for other than reinforced dams. The authors deserve thanks for fearlessly working from the facts and dealing with the actual dam with supposed openings (or cracks) where tension would occur and not with an ideal dam without cracks.

In the usual analysis, the dam is supposed to remain intact. Then, the solution is effected for the dam without weight and omitting the weight of water vertically over it. The latter forces are to be included later. By the authors, they are included from the start, so that it is possible to find the points where tension would be exerted and the sections under compression which alone are to be regarded as effective in computing deflections and stresses.

Similar computations are to be made for cantilevers at other points than the crown and for arches at various elevations. The labor of so many hundreds of trial computations is enormous and looks prohibitive except with a large office force; so that any hints that may reduce this labor may prove welcome.

By shifting the origin to the "elastic center", the number of terms in Equations (24) to (29) is greatly reduced and the solution more easily effected. Otherwise, in place of normal loads, equivalent vertical and horizontal loads can be taken. The solution, where such loads are taken as unit loads for symmetrical arches, has been given by Charles S. Whitney, M. Am. Soc. C. E.* Very simple formulas are derived for finding the reactions when bending only is considered, the effects of direct thrust being treated separately. The writer† derived very simple formulas for the reactions for both vertical and horizontal loads (including symmetrical horizontal loads) where the chances for mistake seem to be reduced to a minimum. However, the graphical method given by Mr. Whitney‡ for finding reactions and deflections at any point will especially appeal to the computer as giving quickly the quantities sought.

If a uniform thickness of a symmetrical arch is assumed, it is possible to derive formulas for moments, thrusts, and crown deflections for two symmetrically placed single loads, acting normally to the extrados, and then, for any assumed continuous loading, to sum the results for the whole load. This is probably the quickest method of effecting a solution for an arch nowhere subjected to appreciable tension; but it fails otherwise. When the thickness of the arch varies, the arch will have to be divided into a number of parts and a method of summation will have to be used.§ For the unsymmetrical arch, Melan's method may suffice.

* "Design of Symmetrical Concrete Arches," *Transactions*, Am. Soc. C. E., Vol. 88 (1925), p. 931.

† *Loc. cit.*, p. 1030.

‡ *Loc. cit.*, p. 957, and Figs. 5 and 16.

§ See *Transactions*, Am. Soc. C. E., Vol. 90 (June, 1927), p. 515, for Mr. Jakobsen's proposed solution.

FRED A. NOETZLI,* M. AM. SOC. C. E. (by letter).—Much effort has been spent during recent years to evolve an analysis of arch dams which should accord, as nearly as practicable, with the theoretical principles used in the design of other types of concrete structures.

The authors base their method on the generally accepted assumption that an arch dam is composed of a system of horizontal arches and vertical cantilevers. The partial loads carried by the individual elements are then determined by successive trials until the deflections of the two systems are equal. The principal advance represented by this paper is in the number of vertical sections considered, and the development of practical methods for the required computations.

The fact that this method of analysis originated in the offices of the U. S. Bureau of Reclamation and has been used in the design of several large arch dams, notably the Horse Mesa, Gibson, and Owyhee Dams, should give it much weight.

In certain respects, the same principles were used in the design of the La Jogne Arch Dam,† in Switzerland, although the required load distribution was determined by a different method. For that dam there were investigated nine vertical cantilevers and four horizontal arch elements. The partial loads carried by each element were evaluated by a mathematical process so that the deflections of the arch and cantilever elements were identical at corresponding points of intersection of the two systems. The tension stresses in the La Jogne Dam were limited to 52 lb. per sq. in. (4 kg. per sq. cm.), however, and secondary arches were not considered.

In detailed application the authors' method utilizes assumptions which mark a radical departure from the principles ordinarily used in the design of concrete structures in general, and dams in particular. For instance, the authors frankly assume that high tension stresses and numerous cracks are likely to occur in arch dams under load. By their method of analysis the parts of the arch and cantilever sections presumably cracked or in tension are neglected, and the final analysis of stresses is made by considering the reduced sections, that is, those in compression, only.

When, in 1920,‡ the writer suggested that arch dams designed according to the cylinder formula were likely to develop cracks in both horizontal and vertical planes, criticism arose. It is a source of satisfaction to the writer that his conclusions, as then stated, have since been substantially vindicated by tests.

There is some uncertainty regarding the depth of the open cracks assumed by the authors and the corresponding forces acting in the cracks due to the hydrostatic pressure of the water in the reservoir. Evidently, the cracks on the air side of the dam are not subject to such pressure.

After a crack has developed in the concrete, the adjacent parts, formerly stressed in tension, will immediately be relieved of stress by the formation

* Cons. Hydr. Engr., Los Angeles, Calif.

† Dr. A. Stucky, "Etude des Barrages Arques," *Bulletin technique de la Suisse Romande*, Lausanne, 1922.

‡ "Gravity and Arch Action in Curved Dams," *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 1.

of the crack. There is no reason for additional cracks forming in the immediate vicinity of the first one, and the thickness, area, and moment of inertia of the primary and secondary arches are substantially the same, except, of course, in the immediate vicinity of the cracks.

Thus, in an arch dam subjected to load there may be excessive tension stresses at both abutments at the extrados, and at the crown of the intrados, producing cracks at all three points. In the immediate vicinity of the cracks, areas that were plane before bending evidently are no longer plane after bending of the arch. In other words, deformations equivalent to a slight rotation are taking place near the cracks. This has an effect similar to that of "restrained" hinges, and the "hingeless" primary arch with cracks at both abutments and at the crown, therefore, will be converted into a structural element resembling a three-hinged arch.

Secondary arching may occur in structures before cracks have occurred, in accordance with Castigliano's law of "least work". This is observed frequently in tunnel excavations. In some such cases, material has been removed to the surface of the theoretical secondary arch without impairing the strength of the structure.

It has been thoroughly demonstrated that the older and simpler methods of designing thick arch dams were at best only roughly approximate and frequently erroneous. The need of refinements in design was evident, and has led to many studies and much progress in the art. The advances in theory, however, have also served to emphasize the inherent uncertainties involved in the assumptions on which the theory must be applied. In other words, the more designers are able to visualize the mechanics of the thick arch dam, the more they appreciate its complexity and the practical limitations of its accurate stress analysis. In view of these multiplying difficulties, to say nothing of other aspects of the matter, it is perhaps indicated that future efforts should be directed toward alterations in the type of structure, such as would eliminate, in part at least, if possible, some of the uncertainties which are inherent in, and somewhat peculiar to, the thick arch dam. In other words, a too great complexity of structural action may reasonably suggest the desirability of a change in the design in the direction of simplicity.

An effort in this direction, with gratifying success, was made by the writer in the design of the Railroad Canyon Arch Dam, in California, built in 1927-28. The dam is approximately 100 ft. high. It is of the variable radius arch type and has an up-stream slope at the arch crown of 10 on 4. Instead of designing the dam on the ordinary arch principle, then assuming cracks in the concrete, and neglecting the concrete cracked or in tension, as in the authors' method, the writer shaped the arches, as constructed, directly along the outline of the "secondary" arches, thereby eliminating tension, at least in any appreciable amount, together with the numerous attendant uncertainties. Incidentally, this design saved the concrete which, under the assumption of either "reduced" or "secondary" arches as portions of thick arches, would have been placed in the structure without any considerable increase in its strength.

FREDRIK VOGT,* ASSOC. M. AM. SOC. C. E. (by letter).—The authors deserve credit for having brought this method of calculation to discussion, since it may lead to improvements in the design of dams. Their main discussion is confined to the calculation of stresses under certain conditions, and the assumptions made are only partly dealt with. It seems to the writer that a discussion of the assumptions is the more important phase. A rough approximation based on correct assumptions with regard to shrinkage, temperature changes, yielding of foundation, etc., is more valuable than a highly refined computation based on incorrect assumptions. However, it is easily understood that the authors wished to limit the problem.

Ignoring all parts of the structure in which tensile stresses occur in order to calculate the greatest compression stresses seems at first glance to be on the side of safety. However, if at first a single crack is opened, due to high tension, the tensile stresses at the neighboring sections are at once reduced to such small magnitudes, that the occurrence of more cracks is not probable. A cantilever with only one horizontal crack at the foundation joint is less flexible than a cantilever without any tensile strength.† In order to obtain the necessary deflection of the arch rings, the eccentricity of the load on the cantilevers and the compression stresses at the toe must be greater if only a single crack is formed than if all the tensile strength can be neglected.

When a large part of the arch section is either affected by tensile stresses, or is cracked, the material is only partly utilized. However, when the dam is constructed in the usual manner, it is impossible to avoid this waste of material. The writer has worked on the problem of how to make the cylinder formula valid for arch dams, thus adding to the ability of the arches to take loads without increasing the stresses in order to improve the safety factor or to cut down the dimensions of the structure. He thinks that this result can be obtained by using a multiple-pressure grouting system, dividing a number of the contraction joints into closed compartments by vertical and horizontal water-stops, so that each of the compartments can be grouted under separate pressure. By such a system it is possible to apply initial stresses in the dam, distributed in such a way that the resulting stresses due to load, shrinkage, cooling, and grouting are approximately equalized over the section. Not only will the arch effect be improved by making the cylinder formula valid in this way, but by pressing grout into closed compartments an actual pressure grouting also can be obtained in order to reduce the stresses in the cantilevers and at least to some extent convert the tensile stresses into compression. The writer agrees with the authors that dams constructed as at present, without any actual pressure grouting, should be designed with constant radius where the canyon is wide as compared with the height of the dam. However, by applying a multiple-pressure grouting, the use of the constant-angle arch principle can also be made suitable for dams in wide canyons, since the cantilever load can be checked. Furthermore, the use of partly open joints of, say, ½-in. to 1-in. opening, makes it possible to cool the dam by circulating water in order

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† Some experiments on this subject have been made by E. Aarskog, Civ. Engr., Dept. of Public Roads, Oslo, Norway.

to get rid of the chemical heat before grouting and to apply the grouting without waiting for a natural opening of the joints due to shrinkage and cooling.

The method of calculation in itself does not seem quite adequate until three factors affecting the load distribution are taken into account: First, the bending of the cantilevers due to the lateral expansion which is caused by arch stresses and *vice versa;* second, the effect of stiffness against torsion; and, third, the effect of stiffness against tangential displacements in the structure.

At the crown, the water-load produces high compression stresses at the up-stream face and low compression on tensile stresses at the down-stream face. The lateral expansion due to these stresses tends to bend the cantilever down stream, thus reducing the bending stresses in the cantilevers. After the arch stresses are computed, this bending can easily be calculated for all cantilevers. The influence of lateral expansion—due to stresses in the cantilevers from water-load only—on the arch deflection will usually be negligible.

In two cases the writer has found that the stiffness against torsion of cantilevers and arch rings reduces the stresses slightly, but, since this effect is of little importance, at least for the examples dealt with, and as they are difficult to compute, he agrees that it is best to neglect it. Perhaps, the torsion may be of importance for very thick arch dams.

FIG. 37.

The stiffness against tangential displacements, on the other hand, may be of material importance in some cases, and when taken into account may cause a considerable redistribution of the stresses. The story of two dam failures described in the technical press* gives a convincing illustration of the importance of this stiffness. In one case the rock abutment of the arch was washed out; in the other case a gravity abutment turned over, so that both arches were without support for more than one-half their height. However, the arch itself did not fail in either case. If the dams were considered as consisting of arch rings and cantilevers, as discussed by the authors, and the arch rings were without support and were unable to take any load in the upper half of the height, then the cantilevers would be certain to fail since the sections were very slender. In the second case, the dam would have to stand the total water pressure without support from the abutment for some time before the water was lowered. Moreover, it would have to stand the shaking due to the failure of the abutment. Only factors other than those dealt with in the paper could save the arches, and among these the stiffness against the tangential displacements may be the most important. It also can be stated that the

* *Engineering News-Record*, Vol. 2, 1926, p. 616.

arch pressure in this case has been directed down to the remaining foundation, as indicated in Fig. 37.

In order to check the importance of this stiffness, in the case of dams with abutments which have not failed, the writer made tests on a small rubber model in Trondjhem, Norway. (See Figs. 38 and 39.) The model itself and the foundation were both of soft rubber mounted in a wooden frame and loaded with water. Radial and tangential displacements were measured for seven vertical and six horizontal sections (see Fig. 40). The diagrams show at once that there is a twisting at the foundation due to a bending moment which verifies the calculations made by the writer* for the yielding of elastic foundations. Furthermore, tangential displacements are found to have about the expected magnitude, and they cannot occur except in connection with tangential shear forces, which tend to increase the stiffness of the arch rings against deflection. All the different kinds of rubber considered showed a high "time effect" and did not give an entirely straight stress-strain diagram. Furthermore, Poisson's ratio was about 1:2.2 for the rubber chosen for the experiment, instead of 1:6 or 1:8 for concrete, and this makes some difference in the coefficients for the shear deformation.

Due to the time effect, it was difficult to get accurate measurements, and these may be taken mainly as qualitative and not quantitative. The radial deflection was measured directly by means of micrometer screws with electric contacts. The tangential displacements were converted into radial displacements by means of a lever system, and were measured by means of the same screws. These screws were mounted on a turnable post, one screw at each elevation for which the displacements were to be measured. The observations were read to the nearest 0.0002 in. and the maximum deflection was about 0.06 in. The stiffness against tangential displacements is of importance mainly at dams with a great ratio between span and height, that is, for all the dams referred to in the paper.

In a circular arch ring of constant thickness with a uniform distribution of the load, the computation of tangential displacements is relatively simple, and this case will be described in order to give an idea of its importance. To simplify the problem the foundation is assumed to be unyielding. By means of a trial load method the tangential displacements can be taken into account more accurately whether or not the foundation is assumed to be firm or yielding.† The matter of making the computations is mainly a question of patience and time.

The method of computing the radial deflection in arches is well known and the same method can also be applied to the calculation of the tangential displacements. Therefore, the writer gives the final formulas without derivations.

* "Ueber die Berechnung der Fundamentdeformation," *Det Norske Videnskapsakademi*, Oslo, Norway, 1925.

† See the writer's discussion of the paper by B. F. Jakobsen, M. Am. Soc. C. E., entitled "Stresses in Thick Arches of Dams," *Transactions*, Am. Soc. C. E., Vol. 90 (1927), p. 475, and the Report on the Stevenson Creek Test Dam, *Proceedings*, Pt. 3, Am. Soc. C. E., May, 1928, p. 129, where it is stated that the writer's formulas for the foundation yielding are checked by the experiments. These formulas are also included in the trial load method as used at present by the U. S. Bureau of Reclamation, see the discussion by J. L. Savage and Ivan E. Houk, Members, Am. Soc. C. E., p. 1289.

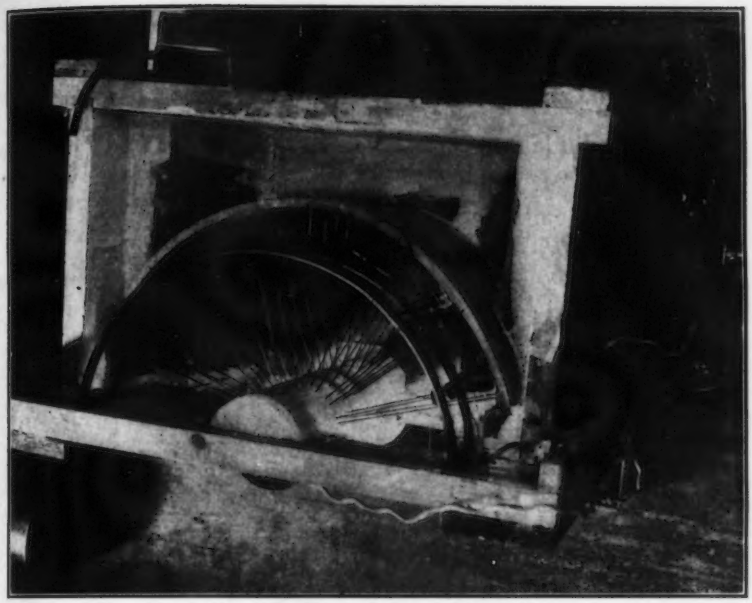


FIG. 38.—TOP VIEW OF RUBBER MODEL DAM.

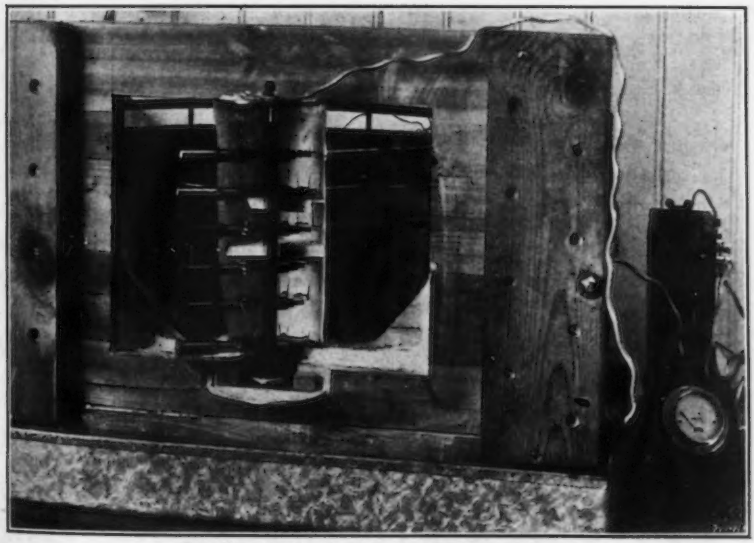
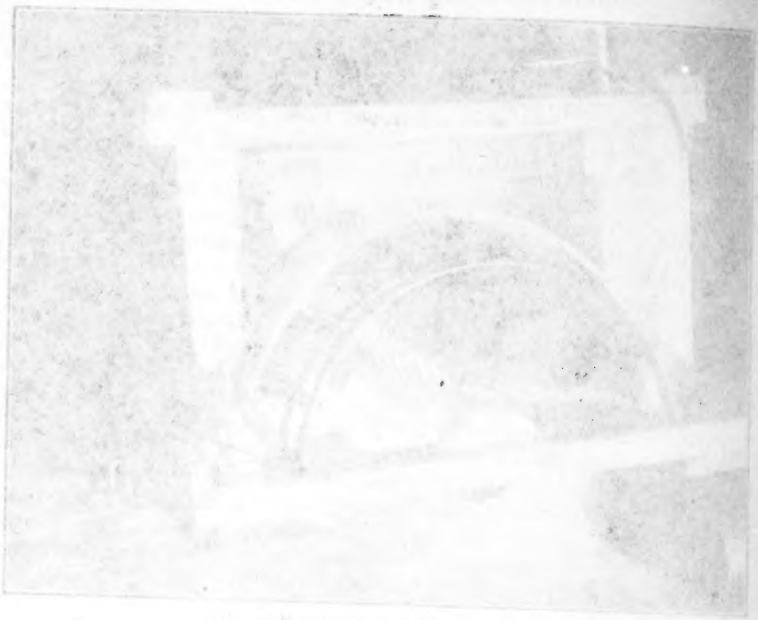
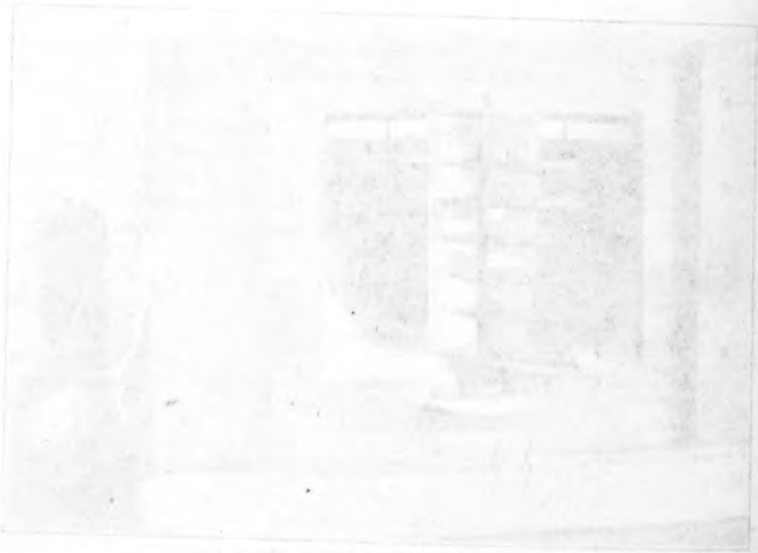


FIG. 39.—FRONT VIEW OF RUBBER MODEL DAM.





The radial deflection at the crown of the arch due to a uniform load, p_0 (Fig. 41), is:

$$\Delta r_0 = \lambda_0 \rho \frac{p_0 r_n r_e}{E t} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (115)$$

in which, λ_0 and ρ are two coefficients;* r_n = the radius to the neutral axis; r_e = the radius to the up-stream face; t = the thickness of the arch; and E = the modulus of elasticity.

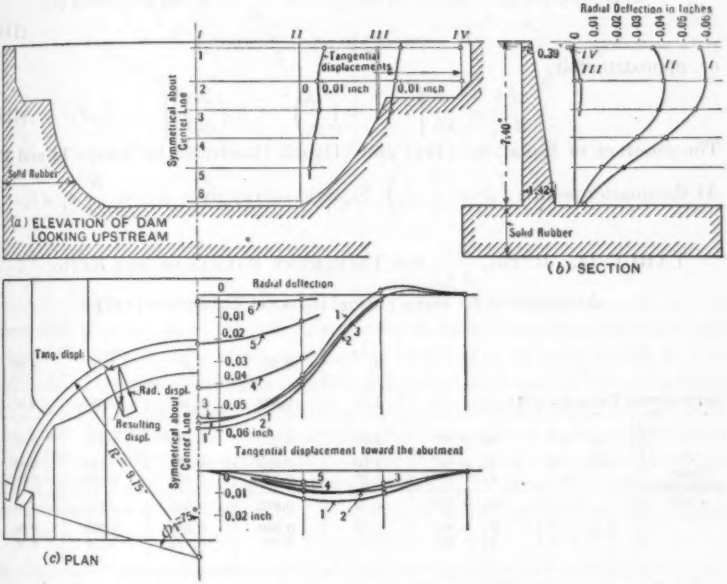


FIG. 40.—MODEL DAM MADE OF SOFT RUBBER. DEFLECTION DUE TO WATER-LOAD.

The radial deflection, Δr, at any point of the arch compared with the deflection at the crown is given by Equation (116):

$$\frac{\Delta r}{\Delta r_0} = \frac{a[\phi_1 (1 - \cos\phi_1 \cos\phi) - \phi \sin\phi \sin\phi_1] - b[\cos\phi - \cos\phi_1]\sin\phi_1}{(1 - \cos\phi_1)(a\,\phi_1 - b \sin\phi_1)} \quad (116)$$

in which,

$$a = 1 + \frac{1}{3}\left(\frac{\kappa E}{G} + 1\right)\left(\frac{t}{2 r_n}\right)^2$$

and,

$$b = 1 - \frac{1}{3}\left(\frac{\kappa E}{G} - 1\right)\left(\frac{t}{2 r_n}\right)^2$$

* See the writer's discussion on the paper by B. F. Jakobsen, M. Am. Soc. C. E., entitled "Stresses in Thick Arches of Dams," *Transactions*, Am. Soc. C. E., Vol. 90 (June, 1927), p. 544, Equations (120), p. 558, and (125), p. 564, and Fig. 46, p. 565.

Equation (116) equals approximately:

$$\frac{\Delta r}{\Delta r_0} = \left[1 - \left(\frac{\phi}{\phi_1}\right)^2\right]^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots (117)$$

in which, ϕ_1 is one-half the central angle, and ϕ, the angular distance from the crown to the point in question, both in radians.

Furthermore, at any point except the crown and the abutments a tangential displacement, Δb, directed toward the abutments takes place, as follows:

$$\frac{\Delta b}{\Delta r_0} = \frac{a\,[\phi \cos \phi \sin \phi_1 - \phi_1 \cos \phi_1 \sin \phi] - 2\,\frac{\sin \phi_1}{\phi_1}\,[\phi_1 \sin \phi - \phi \sin \phi_1]}{(1 - \cos \phi_1)\,(a\,\phi_1 - b \sin \phi_1)} \quad \ldots\ldots (118)$$

or, approximately,

$$\frac{\Delta b}{\Delta r_0} = \frac{\phi}{15}\left[7 - 10\left(\frac{\phi}{\phi_1}\right)^2 + 3\left(\frac{\phi}{\phi_1}\right)^4\right] \quad \ldots\ldots\ldots\ldots\ldots (119)$$

The accuracy of Equations (117) and (119) is illustrated by Tables 14 and 15.

At the quarter-points, $\left(\phi = \frac{1}{2}\,\phi_1\right)$, Equation (119) gives $\Delta b = \frac{5}{32}\,\phi_1\,\Delta r_0$.

TABLE 14.—RATIO, $\frac{\Delta r}{\Delta r_0}$, FOR DIFFERENT VALUES OF THE RATIO, $\frac{\phi}{\phi_1}$, ACCORDING TO EQUATION (116) AND EQUATION (117).

$\frac{\phi}{\phi_1}$ =			0	0.25	0.5	0.75	1.00
Approximate Equation (117)			1.00	0.879	0.563	0.191	0.00
Exact Equation (116)	$\frac{t}{2r_n} = 0$	$\phi_1 = 90°$	1.00	0.870	0.540	0.175	0.00
		$\phi_1 = 60°$	1.00	0.875	0.552	0.184	0.00
		$\phi_1 = 30°$	1.00	0.879	0.559	0.190	0.00
	$\frac{t}{2r_n} = 0.1$	$\phi_1 = 90°$	1.00	0.872	0.545	0.181	0.00
	$\frac{t}{2r_n} = 0.1$	$\phi_1 = 60°$	1.00	0.880	0.569	0.202	0.00
	$= 0.1$	$\phi_1 = 30°$	1.00	0.895	0.611	0.258	0.00

TABLE 15.—RATIO, $\frac{\Delta b}{\Delta r_0}$, FOR DIFFERENT VALUES OF THE RATIO, $\frac{\phi}{\phi_1}$, ACCORDING TO EQUATION (118) AND EQUATION (119).

	$\frac{\phi}{\phi_1}$ =		0	0.25	0.50	0.75	1.00
$\frac{t}{2r_n} = 0$	$\phi_1 = 90°$	Exact	0	0.171	0.247	0.181	0
		Approximate	0	0.167	0.245	0.183	0
	$\phi_1 = 60°$	Exact	0	0.113	0.164	0.121	0
		Approximate	0	0.111	0.164	0.122	0
	$\phi_1 = 30°$	Exact	0	0.054	0.082	0.062	0
		Approximate	0	0.056	0.082	0.061	0
$\frac{t}{2r_n} = 0.1$	$\phi_1 = 90°$	Exact	0	0.171	0.248	0.182	0
		Approximate	0	0.167	0.245	0.183	0
	$\phi_1 = 60°$	Exact	0	0.113	0.166	0.124	0
		Approximate	0	0.111	0.164	0.122	0
	$\phi_1 = 30°$	Exact	0	0.054	0.084	0.067	0
		Approximate	0	0.056	0.082	0.061	0

For instance, if one-half the central angle, ϕ_1, is equal to 1.0472 radians (60°), the tangential displacement midway between the crown and the abutment is found to be about one-sixth the radial deflection at the center.

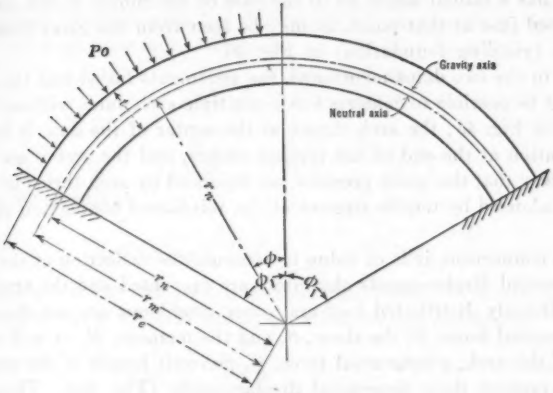


FIG. 41.

If the dam were divided by contraction joints very closely spaced, and if there were no friction in the joints, there would be no resistance against such tangential displacements, since each section would then be very flexible in this direction. However, the space between the contraction joints for most dams (very large dams excepted) is usually greater than the thickness of the dam, and hence the stiffness against tangential displacements of each section taken separately is greater than against radial deflections. Then, too, there is always a large amount of friction (due to the arch thrust) to resist the sliding of the two sections relative to each other. Especially if the joints are grouted, it must

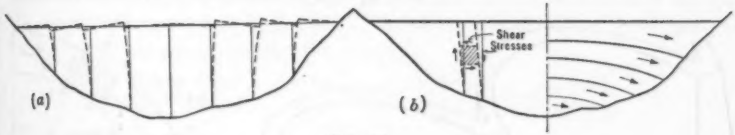


FIG. 42.

be assumed that this friction is great enough to make the dam act as a monolithic structure. If the sections can slide relative to each other, a tangential displacement can be produced by the bending of the sections, as shown in Fig. 42(*a*); but if the sections cannot slide relatively, the tangential displacement can be produced only as a shear deformation, each square being limited by vertical radial planes and horizontal planes must be deformed as shown in Fig. 42(*b*). This diagram also shows the shear stresses producing the deformation. If the bending deformation is taken away, and only the shear deformation remains, a relatively large force is needed to produce a certain displacement. In other words, such a condition makes a dam very resistant to tangential displacements.

If this shear is combined with the arch thrust, the direction of the principal stresses may be about as shown on the right side of Fig. 42(*b*); the arch thrust is bent downward in an inclined direction at the abutments. If the abutment has a salient angle, as in the case of the rubber model, the thrust is concentrated just at that point, as may be seen from the great tangential displacement (yielding foundation) in Fig. 40.

Since, in the two dams mentioned, the abutments failed and the arches did not, it may be possible to construct safe multiple-arch dams without buttresses, as shown in Fig. 43; the arch thrust at the center of the dam is led down to the foundation at the end of the vertical arches, and the arches are connected in such a way that the water pressure not balanced by arch thrust or cantilever action is balanced by tensile stresses at the reinforced connection between the arches.

In this connection, it is of value to compute the deflection of the arch ring if all tangential displacements absolutely are prevented and the arch is loaded with a uniformly distributed load and other conditions are not changed. Besides the normal force, N, the shear, S, and the moment, M, at both ends of an element of the arch, a tangential force, Q, per unit length of the arch axis, is needed to prevent these tangential displacements (Fig. 44). This force, Q, is the resultant of the tangential shear at the upper and lower boundary of the arch ring. The thickness measured square on the plane of Fig. 44 is assumed to be equal to the unit. Three conditions are necessary in order that the hatched element in Fig. 44 shall be in equilibrium:

$$d\,N + Q\,r_n\,d\,\phi = 0 \quad \ldots\ldots (120)$$

$$N\,d\,\phi + d\,S - p_1\,r_e\,d\,\phi = 0 \quad \ldots\ldots (121)$$

$$S\,r_n\,d\,\phi - d\,M = 0 \quad \ldots\ldots (122)$$

The total radial deflection, y, can be considered as consisting of the deflection, y_1, due to the moment, M, and y_2 due to the shear, S:

$$y = y_1 + y_2 \quad \ldots\ldots (123)$$

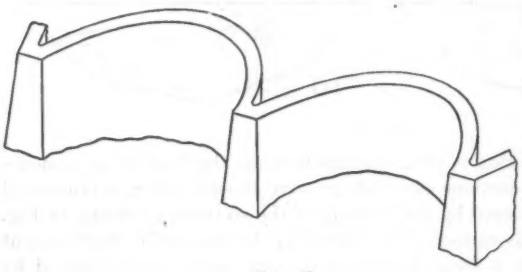


FIG. 43.

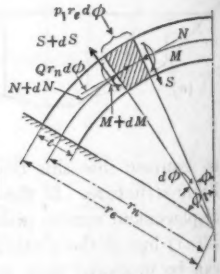


FIG. 44.

The unit axial compression, u, of the arch is determined directly by the radial deflection, y, since all tangential displacements are assumed to be prevented and only radial deflections are allowed:

$$u = \frac{y}{r_n} \quad \ldots\ldots (124)$$

Furthermore, the axial compression can be expressed by the normal force:

$$N = u\,E\,\frac{t}{\rho} = y\,E\,\frac{t}{\rho\,r_n} \quad \ldots\ldots (125)^*$$

The usual formulas give the derivatives of the terms, y_1 and y_2, which together constitute the total deflection:

$$\frac{d^2\,y_1}{r_n{}^2\,d\,\phi^2} = \frac{M}{E\,I} = \frac{12\,M}{E\,t^3} \quad \ldots\ldots (126)\dagger$$

$$\frac{d\,y_2}{r_n\,d\,\phi} = -\frac{\kappa\,S}{G\,F} = -\frac{\kappa\,S}{G\,t} \quad \ldots\ldots (127)$$

By using the notations,

$$y_1 = \rho\,\frac{p_1\,r_e\,r_n}{E\,t};\ a^4 = \frac{3\,r_n{}^2}{\rho\,t^2};\ b^2 = \frac{1}{4\,\rho}\,\frac{\kappa\,E}{G} \quad \ldots\ldots (128)$$

an elimination of the different unknown magnitudes by means of Equations (120) to (128) gives the differential equation for the deflection,

$$\frac{d^4\,y}{d\,\phi^4} - 4\,b^2\,\frac{d^2\,y}{d\,\phi^2} + 4\,a^4\,y = 4\,a^4\,y_1 \quad \ldots\ldots (129)$$

The moment and shear can be expressed by the deflection:

$$M = \frac{E\,t}{4\,a^4\,\rho}\left[\frac{d^2\,y}{d\,\phi^2} - 4\,b^2\,(y - y_1)\right] \quad \ldots\ldots (130)$$

$$S = \frac{E\,t}{4\,r_n\,a^4\,\rho}\left[\frac{d^3\,y}{d\,\phi^3} - 4\,b^2\,\frac{d\,y}{d\,\phi}\right] \quad \ldots\ldots (131)$$

From the symmetry of the arch it follows that at the crown ($\phi = 0$):

$$\frac{d\,y}{d\,\phi} = \frac{d^3\,y}{d\,\phi^3} = 0 \quad \ldots\ldots (132)$$

At the abutments ($\phi = \phi_1$), $y = 0$. Furthermore, the moment in itself produces no change in direction for the arch axis at the abutment if the dam is assumed to be fixed at both ends. The change in direction there can only be caused by the shear, so that for $\phi = \phi_1$:

$$\frac{d\,y}{d\,\phi} = \frac{d\,y_2}{d\,\phi} = -\frac{\kappa\,r_n}{G\,t}\,S = -\frac{b^2}{a^4}\left[\frac{d^3\,y}{d\,\phi^3} - 4\,b^2\,\frac{d\,y}{d\,\phi}\right] \quad \ldots\ldots (133)$$

or,

$$b^2\,\frac{d^3\,y}{d\,\phi^3} + (a^4 - 4\,b^4)\,\frac{d\,y}{d\,\phi} = 0 \quad \ldots\ldots (134)$$

The general differential equation is satisfied by the expression:

$$y = y_1 + C_1\,e^{\alpha\phi}\sin\beta\,\phi + C_2\,e^{\alpha\phi}\cos\beta\,\phi + C_3\,e^{-\alpha\phi}\sin\beta\,\phi + C_4\,e^{-\alpha\phi}\cos\beta\,\phi \quad \ldots\ldots (135)$$

in which,

$$\alpha = \sqrt{a^2 + b^2} \text{ and } \beta = \sqrt{a^2 - b^2} \quad \ldots\ldots (136)$$

---

* ρ is equal to unity for "thin" arches.

† For thick arches the moment of inertia, $I = \frac{t^3}{12}$, used as in Equation (126) does not give absolutely exact values for the bending, but the difference is without any importance, as shown by B. F. Jakobsen, M. Am. Soc. C. E., in his paper, "Stresses in Thick Arches of Dams," *Transactions*, Am. Soc. C. E., Vol. 90 (June, 1927), p. 475.

and C_1 to C_4 are four integration constants. These constants must be determined by the conditions at $\phi = 0$, and $\phi = \phi_1$, previously discussed. The single solution which agrees with these conditions is:

$$y = y_1 [1 - A \sinh \alpha \phi \sin \beta \phi - B \cosh \alpha \phi \cos \beta \phi] \ldots\ldots (137)$$

in which,

$$A = \frac{\alpha (3 \beta^2 - \alpha^2) \sinh \alpha \phi_1 \cos \beta \phi_1 - \beta (3 \alpha^2 - \beta^2) \cosh \alpha \phi_1 \sin \beta \phi_1}{\beta (3 \alpha^2 - \beta^2) \sinh \alpha \phi_1 \cosh \alpha \phi_1 + 2 (3 \beta^2 - \alpha^2) \sin \beta \phi_1 \cos \beta \phi_1} \ldots (138)$$

and,

$$B = \frac{\beta (3 \alpha^2 - \beta^2) \sinh \alpha \phi_1 \cos \beta \phi_1 + \alpha (3 \beta^2 - \alpha^2) \cosh \alpha \phi_1 \sin \beta \phi_1}{\beta (3 \alpha^2 - \beta^2) \sinh \alpha \phi_1 \cosh \alpha \phi_1 + \alpha (3 \beta^2 - \alpha^2) \sin \beta \phi_1 \cos \beta \phi_1} \ldots (139)$$

At the crest of the arch, $\phi = 0$, the deflection is found from Equation (137):

$$\Delta r_1 = y_1 [1 - B] = \lambda_1 \rho \frac{p_1 r_n r_e}{E t} \ldots\ldots (140)$$

in which,

$$\lambda_1 = 1 - B \ldots\ldots (141)$$

and y_1 is the deflection according to the cylinder formula.

As may be seen from this development, the final formula (Equation (140)) for the deflection at the crown of the arch is the same whether or not the arch ring is prevented from tangential displacements, except that the coefficient, λ_0, in Equation (115) must be replaced by λ_1, if these displacements are prevented. By calculating the coefficients, λ_0 and λ_1, the influence of the stiffness against tangential displacements on the stiffness against radial displacements can be judged. For instance, for an arch with a thickness, t, equal to one-twentieth the radius, r_n, of the neutral axis, Equation (115) shows that if $\frac{\kappa E}{G} = 2.8$, $\lambda_1 = 1.075$ if $\phi_1 = 30°$ and $\lambda_1 = 0.997$ if $\phi_1 = 60$ degrees.

The corresponding values for the coefficient, λ_0, are 1.67 and 1.87.* This means that if the dam is absolutely rigid against tangential displacements, the stiffness of the arch rings will be increased in a ratio of 1.075 to 1.67, or 55% for the first case and, correspondingly, 88% for the second case. Of course, the dam is not absolutely rigid against such displacements; it is only very rigid if the ratio between its span and height is relatively great. On the whole, the shear deformation in the arch does not influence the value of the coefficient, λ_1, very much. To neglect the shear deformation in the development given previously is the same as putting $\frac{\kappa E}{G}$ equal to zero, and then the coefficient, λ_1, is simplified as follows:

$$\lambda_1 = 1 - B = \frac{(\sinh w - \sinh w)(\cosh w - \cosh w)}{\sinh w \cosh w + \sin w \cos w}$$

$$= \frac{w^4}{6} \left[1 - \frac{163}{1\,260} w^4 + \ldots\right] \ldots\ldots (142)$$

in which,

$$w = a \phi_1 = \phi_1 \sqrt[4]{\frac{3 r_n^2}{\rho t^2}} \ldots\ldots (143)$$

* *Transactions*, Am. Soc. C. E., Vol. 90 (June, 1927), p. 565, Fig. 46.

This formula for the coefficient, λ_1, as a function of w is demonstrated graphically in Fig. 45. If the arch is prevented from tangential displacements by means of tangential reactions (shear forces), the arch thrust at any point is proportional to the deflection as given by Equation (125). Therefore, at the abutments the thrust in such an arch is equal to zero; or, in other words, all the thrust is taken up by the tangential shear forces between the crown and the abutment.

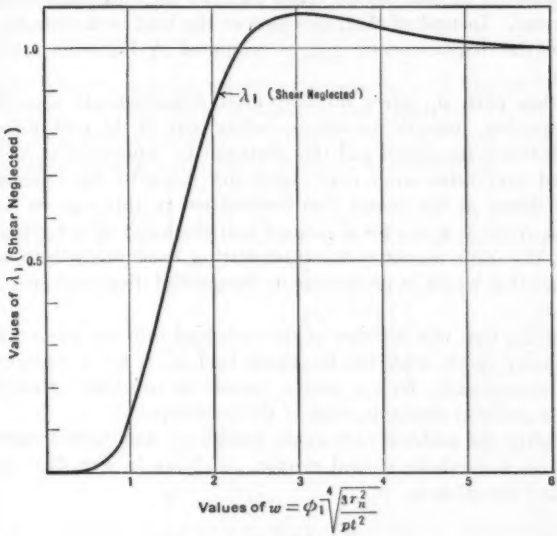


FIG. 45.

In an actual dam this should mean that the thrust in the upper arch rings is transferred to the lower arch rings by the shear forces between the different rings. Since the structure may allow some tangential displacements, not all the arch thrust is transmitted in this way, but only a part of it. How great this is depends mainly on the ratio between span and height and the other dimensions of the dam. Imagine, for instance, that the load, p_a, taken by arch action is divided into two parts, one part, p_0, being carried by the horizontal arch thrust over to the abutments, and another part, p_1, being transferred by tangential shear over to the other arch rings. Together these parts of the load give the total arch load:

$$p_a = p_0 + p_1 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (144)$$

The first part of the load contributes to the deflection at the crown,

$$\Delta r_0 = \lambda_0 \rho \frac{p_0 r_e r_n}{E t} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (145)$$

and the second part contributes,

$$\Delta r_1 = \lambda_1 \rho \frac{p_1 r_e r_n}{E t} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (146)$$

or, together,

$$\Delta r = \Delta r_0 + \Delta r_1 = \lambda \rho \frac{p_a r_e r_n}{E t} \quad \ldots\ldots (147)$$

in which,

$$\lambda = \frac{\lambda_0 p_0 + \lambda_1 p_1}{p_a} \quad \ldots\ldots (148)$$

The first part of the load, p_0, gives an arch thrust carried horizontally over to the abutments as mentioned, which signifies no tangential forces between the arch rings. Instead of that, this part of the load is inseparably connected with tangential displacements, Δb, as computed by Equation (118) or Equation (119).

The other part, p_1, gives no tangential displacements according to the assumptions; but, instead, the corresponding part of the arch thrust must be taken up between the crown and the abutment by tangential shear forces, and transmitted over other arch rings vertically down to the foundation. The total arch thrust at the center line transmitted in this way on each half of the arch is, $N = \lambda_1 p_1 r_e$, for a ring of unit thickness in a vertical direction. Of course, the shear necessary for transmitting these forces between the arch rings is just that which is producing the tangential displacements, Δb, in the cantilevers.

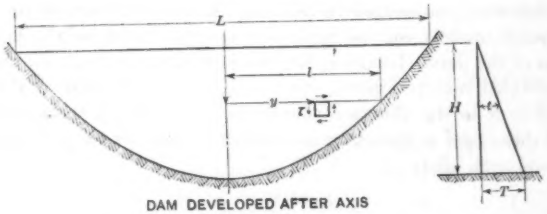
It is certain that this division of the arch load into two parts can only give an approximate result, since the total arch load, p_a, is not a constant along the arch and, consequently, both p_0 and p_1 cannot be constant. However, it may be useful in order to obtain an idea of the conditions.

To simplify the problem still more, consider a dam with triangular cross-section across a parabolic shaped canyon, as shown in Fig. 46. One-half the length at any elevation is,

$$l = \frac{L}{2} \sqrt{1 - \frac{x}{H}} = \phi_1 r_n \quad \ldots\ldots (149)$$

and the thickness is,

$$t = \frac{T}{H} x \quad \ldots\ldots (150)$$



DAM DEVELOPED AFTER AXIS

FIG. 46.

Introducing at any point the horizontal distance, y, from the center line, $y = \phi r_n$, the tangential displacement against the abutment (see Equation (119)) may be expressed as:

$$\Delta b = \lambda_0 \rho \frac{p_0 r_e r_n}{E t} \frac{y}{15 r_n} \left[7 - 10 \left(\frac{y}{l} \right)^2 + 3 \left(\frac{y}{l} \right)^4 \right] \quad \ldots\ldots (151)$$

The shear stresses, horizontal and vertical, in the plan of the dam are $\tau = -G\frac{\delta \Delta b}{\delta x}$, in which, G is the shearing modulus. When the thickness of the dam is t, the difference in shearing force per unit length of the axis between the lower and upper limit of an arch ring is $\frac{\delta (\tau t)}{\delta x}$, and the total difference in this shearing force from crown to abutment must be equal to the thrust from the load, p_1:

$$\lambda_1 p_1 r_e = \int_0^l \frac{\delta (\tau t)}{\delta x} d y \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (152)$$

For the integration assume that the product, $\lambda_0 \rho$, is approximately constant; the variation is negligible, at least for the upper part of the arch. By substituting the different expressions given for l, t, Δb, and τ, in Equation (152), the ratio between p_1 and p_0 will be found to be equal to:

$$\frac{p_1}{p_0} = \frac{\lambda_0 \rho}{120 \lambda_1} \frac{G}{E} \frac{L^2}{H^2} \left[\frac{1 + 3\frac{H}{x}}{1 - \frac{x}{H}} - \frac{3 H^2}{p_0} \left[\left(1 - 3\frac{x}{H}\right) \frac{\delta \left(\frac{\rho_0}{x}\right)}{\delta x} + x \left(1 - \frac{x}{H}\right) \frac{\delta^2 \left(\frac{\rho_0}{x}\right)}{\delta x^2} \right] \right] \quad \ldots\ldots\ldots\ldots\ldots\ldots (153)$$

If Poisson's ratio, $\frac{1}{m}$, is $\frac{1}{6}$, then,

$$\frac{G}{E} = \frac{m}{2 (m + 1)} = \frac{3}{7}$$

and $\frac{\lambda_0 \rho}{\lambda_1}$ may be taken equal to 1.8, an average for the whole upper part of the arch.

A dam with zero thickness at the crest takes zero arch load at the crest, and both p_0 and p_1 must be zero at $x = 0$. This boundary condition is fulfilled if it is assumed that p_0 is proportional to $x\, e^{\frac{x}{H}}$, at least for small values of x. Then the ratio is,

$$\frac{p_1}{p_0} = \frac{\lambda_0 \rho}{120 \lambda_1} \frac{G}{E} \frac{L^2}{H^2} \frac{10 - 3\frac{x}{H} - 3\left(\frac{x}{H}\right)^2}{1 - \frac{x}{H}}$$

or, for $x = 0$,

$$\frac{p_1}{p_0} = \frac{\lambda_0 \rho}{12 \lambda_1} \frac{G}{E} \frac{L^2}{H^2} = 0.0643 \frac{L^2}{H^2} \quad \ldots\ldots\ldots\ldots (154)$$

This function for p_0 cannot be considered valid over its entire height because that would only lead to absurd conditions. At the base all the arch load must be taken by shear since the lowest arch is directly connected with the foundation along the entire length; that is, $p_0 = 0$ and $\frac{p_1}{p_0}$ is infinitely

great. The total arch load is always greater than zero at the bottom if the length profile of the dam is curved. These conditions are fulfilled, for instance, if p_0 is proportional to $x\left(1-\frac{x}{H}\right)$, except for small values of x. It does not matter very much if p_0 is determined by a more complicated function fulfilling the conditions at the limits. Nor does it matter how the transfer between the exponential function at the top and the parabola at some section below is assumed. The main thing is only that p_0 must increase at a rate nearly proportional with x at the top and then must follow any smooth curve down to zero at $x = H$. If p_0 is found by the parabola formula, the ratio is,

$$\frac{p_1}{p_0} = \frac{\lambda_0\,\rho}{60\,\lambda_1}\,\frac{G}{E}\,\frac{L^2}{H^2}\,\frac{3\,\frac{H}{x}-1}{1-\frac{x}{H}} \qquad \text{.....(155)}$$

Table 16 gives a comparison of values obtained for the ratio, $\frac{p_1}{p_0}$ by Equations (154) and (155), respectively.

TABLE 16.—Comparative Values of $\frac{p_1}{p_0}$.

$\frac{x}{H}$.	ρ_0 proportional to $x\,e^{\frac{x}{H}}$.	ρ_0 proportional to $x\left(1-\frac{x}{H}\right)$.
0.	$0.064\left(\frac{L}{H}\right)^2$	.....
0.1	$0.069\left(\frac{L}{H}\right)^2$	$0.414\left(\frac{L}{H}\right)^2$
0.2	$0.075\left(\frac{L}{H}\right)^2$	$0.225\left(\frac{L}{H}\right)^2$
0.3	.....	$0.165\left(\frac{L}{H}\right)^2$
0.4	.....	$0.139\left(\frac{L}{H}\right)^2$
0.5	.....	$0.129\left(\frac{L}{H}\right)^2$
0.6	.....	$0.129\left(\frac{L}{H}\right)^2$
0.7	.....	$0.141\left(\frac{L}{H}\right)^2$
0.8	.....	$0.177\left(\frac{L}{H}\right)^2$
0.9	.....	$0.300\left(\frac{L}{H}\right)^2$
1.0	.....	Infinite

As will be seen, the ratio is not a constant value, but in the central part of the dam the variation is not great. The same result will be found if other reasonable assumptions for p_0 are made. The value depends mainly on the ratio between the length, L, and the height, H.

A similar analysis can be easily made for other cross-sections and canyon profiles. Some difficulties may result if the arches near the crest are designed too strong, because this may sometimes lead to a negative value for the part, p_1, of the load. However, the fact remains that a certain fraction of the arch

load, the fraction depending mainly on the ratio, $\frac{L^2}{H}$, must be transmitted vertically down to the foundation by tangential shear between the arch rings. This fraction is found to be smaller than in the case of a triangular section if the thickness of the dam is the same for different elevations, as in the Stevenson Creek Dam. Since practical dams always have a more or less triangular-shaped cross-section, the writer has used the following formula in the computation of stresses in some dams:

$$R = \frac{p_1}{p_0} = \frac{1}{10}\left(\frac{L}{H}\right)^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (156)$$

It is possible that,

$$R = \frac{p_1}{p_0} = \frac{1}{8}\left(\frac{L}{H}\right)^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (157)$$

would give a better average.

The stiffness against tangential shear deformation is taken into account in this way, at least approximately, and without any noticeable increase in the numerical calculation. Equation (148) is then found to be:

$$\lambda = \frac{\lambda_0 + R\,\lambda_1}{1 + R} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (158)$$

Both λ_0 and λ_1 can be taken directly from the diagrams, and R is the ratio given by Equation (156) or Equation (157).

For instance, for the Gibson Dam, the length is about 4.5 times the height, the ratio between the coefficients, λ_0 and λ_1, is about 1.8 for the greatest part of the dam, and the stiffness of the arch rings against deflection in this way is found to be increased by about 40 to 45%; that is, with the same deflection, the arch load is increased by 40 to 45% due to this tangential stiffness. That, of course, will also change the load distribution between the arches and the cantilevers. However, the change in load will be less than 40% since the deflection also will be changed.

For arch dams in narrow canyons, the influence will be much less. The tangential stiffness is only of importance at wide dam sites, in which case it is a factor that cannot be neglected in any calculation of stresses. The next question is how to take it into account (at least, approximately) in a way that will not cause too much additional work.

Where only the deflection of one cantilever at the center is compared with the deflection of the arch rings, and the load on each arch ring is assumed to be uniformly distributed, the use of the coefficient, λ_1 (Equation (158)), instead of λ_0, at once gives the solution. As far as actual arch dams with approximately triangular cross-section and large central angles constructed in U-shaped canyons are concerned, the writer has found that these very simplified assumptions give the bending stresses in the highest cantilevers within a few per cent., and even give the arch stresses without very great errors. The distribution of load for that case may be found in one to two days instead of one to two months by the complete trial load method.

Where it is found necessary to take the deflection of more than the central cantilever into account in the computations, it may be convenient to use the following procedure: First, compute the approximate division of load by tak-

ing only the central cantilever into account as before, and by the arch-load diagram and the approximate division of the arch load in the two components, p_0 and p_1, calculate the approximate values of the horizontal arch thrust and the tangential shear force for a number of points at different vertical planes of the dam. By these values for arch thrust and shear the direction of the true thrust can be found in the same way as the principal stresses in an infinitely small cube. The slope of this direction may be ψ. Then,

$$\tan 2\psi = 2 \frac{\text{shear per unit length of arch}}{\text{thrust per unit height — vertical force per unit length of arch}}$$

After the declination of the thrust due to water pressure is found, at least approximately, the trajectories of thrust are plotted and the dam is divided into a number of arch rings limited by these trajectories as shown in Fig. 47. Between these arch rings no shear of any importance can occur, and they may be used for analysis instead of the horizontal arch rings used by the authors. The vertical component of the inclined arch thrust transfers part of the weight of the cantilevers at the center to the abutments of the different arch rings. These vertical forces can be computed directly from the horizontal forces, since the resultant at any point must follow the trajectories. The complication in the numerical computations then is mainly reduced to the use of arch rings of variable thickness, both vertically and horizontally, instead of rings with only the horizontal thickness being variable. The yielding of the foundation may be taken into account in the same manner as for the other arch rings.*

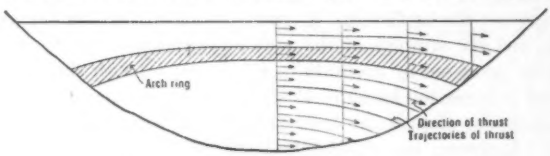


FIG. 47.

The results of the analysis will be changed as follows:

1.—The load taken by the arches is increased and the load taken by the cantilevers reduced, which reduces the bending stresses in the cantilevers.
2.—Part of the weight of the dam at the center will be carried by the vertical components of the arch thrust, and transmitted to either side of the dam. This reduces the compression stresses and increases possible tensile stresses at the central cantilevers and *vice versa* at the cantilevers near the abutments.
3.—The actual arch rings are much stiffer at the abutments than the horizontal arch rings, since the thickness there is increased both vertically and horizontally from the thickness at the crown, which may give quite a different distribution of the arch stresses.

Some of the dams referred to in the paper are made considerably thicker at the springing line than at the crown. It seems doubtful whether an extended analysis of these problems would justify this detail since the difference already is obtained with constant thickness in horizontal planes by using the true inclined arch rings.

* See discussion by Messrs. Savage and Houk, p. 1289.

J. L. SAVAGE* and IVAN E. HOUK,† MEMBERS, AM. SOC. C. E. (by letter).—Several important changes and improvements have been made by the U. S. Bureau of Reclamation in its methods of analyzing arch dams since this paper was prepared. These modifications are too elaborate to be adequately explained in a brief discussion. However, it seems advisable at this time to state briefly the nature of the changes that have been made and to discuss their effect on the analysis of the Gibson Dam now being built on the Sun River Project of Montana, this being one of the dams mentioned by the authors.

In the original analysis of the Gibson Dam the sides of the cantilevers were assumed to be parallel; the temperature curve shown in Fig. 4, which was established on an incomplete study of limited experimental data, was used as a basis for calculating temperature stresses in the arches; the effect of shear in the arches was neglected; the foundation and abutment rock was considered to be rigid; and no arch action was assumed to take place until the increasing water load had reduced the up-stream deflections of the cantilevers to zero. In the revised calculations, the sides of the cantilevers were taken as radial; a new temperature curve, founded on a careful study of many actual measurements of concrete temperature variations, was used as a basis for calculating temperature stresses in the arches; the effect of shear in the arches was considered; the foundation and abutment deformations were taken into account in both cantilever and arch elements; and transmission of load to the abutments by arch action was assumed to begin as soon as the reservoir begins to fill.

Thus far, the writers have not taken into account the saturation of concrete near the up-stream face of the dam, flow of concrete, effect of Poisson's ratio, transmission of load by tangential shear between arches, effect of vertical shear between cantilevers, effect of variations in the modulus of elasticity, the distribution of stress due to twisting of the horizontal and vertical elements, some of the effects introduced through foundation and abutment deformations, or the errors introduced through the assumption of a linear stress distribution. However, all these conditions are being investigated, and the writers hope, eventually, to be able to estimate the magnitude of some of their effects as regards the stresses within the structure and its safety against failure through any cause.

The fact that the sides of the cantilever should be taken as radial vertical planes instead of parallel vertical planes was recognized at the time the original analysis of the Gibson Dam was made. While it was realized that proper allowance for this condition would tend to increase the bending of the cantilevers under given loads, thus tending to increase the stresses at the down-stream face of the dam, it was not believed that the effect of the correction would be as important as it was later found to be, especially as regards bending where the cantilevers are already cracked. Of course, the increased bending and accompanying increased stress under a given load is not the final effect on the analysis of the structure. The arches must bend over to meet the

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† Research Engr., U. S. Bureau of Reclamation, Denver, Colo.

increased deflections of the cantilevers, and in order to do so they will carry some of the load formerly allotted to the cantilevers. Consequently, the net result of considering the cantilever sides as radial instead of parallel is a new load distribution with slightly increased deflections and stresses.

Fig. 48 shows the increase in down-stream deflection of the crown cantilever at the Gibson Dam, also the increase in tension area, caused by considering the sides of the cantilever as radial instead of parallel, the load being the same in each calculation. It will be noticed that, while the proportionate increase in total tension area is not as great as might be expected, the decrease in width of uncracked base and the increase in down-stream deflection of the cantilever, resulting from the corrected method of calculation, are both important. The latter is seen to be practically double the original deflection at all elevations.

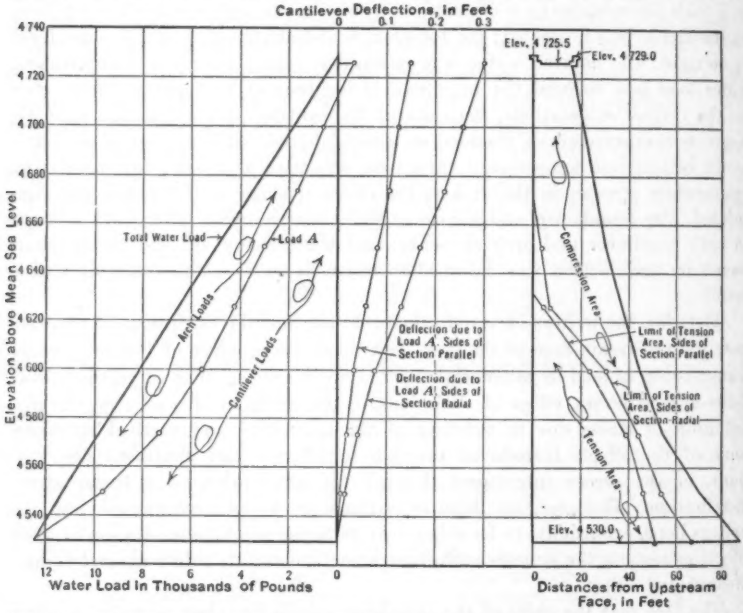


FIG. 48.—CHANGES IN TENSION AREA AND DEFLECTION OF CROWN CANTILEVER AT GIBSON DAM CAUSED BY CONSIDERING RADIAL SIDES OF CANTILEVER.

Fig. 49 compares the original temperature curve of Fig. 4 with the revised curve used in the latest analysis of the Gibson Dam. In determining the revised curve the variations in water temperature at different depths in the reservoir were taken into account, as well as the maximum and minimum air temperatures at the down-stream face of the dam. It will be noticed that the new curve is higher than the old one at all points. Consequently, its use tends to give higher temperature stresses. Since the curve gives the total annual range in concrete temperatures, only one-half the values shown need be used in calculating arch stresses, it being assumed that the contraction joints will be grouted, or closing plugs poured, when the concrete temperatures are equal

to, or less than, the mean annual temperature of the dam. Since the expansion accompanying a rise in temperature works against the water load, only the contraction accompanying the decrease in temperature need be considered. The new curve was not extended farther at the left of the diagram since 15 ft. is the minimum thickness of the Gibson Dam.

The effect of considering shear in the arch calculations was found to be practically negligible in the case of long thin arches, but to increase as the ratio of $\frac{l}{r}$ increased, becoming very important in the case of comparatively short, thick arches. For some of the thick arches analyzed, the deflections

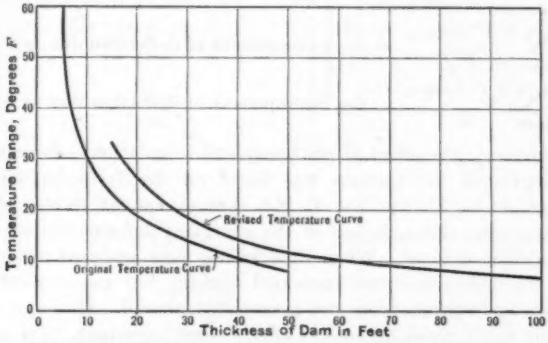


FIG. 49.—COMPARISON OF ORIGINAL AND REVISED TEMPERATURE CURVES USED IN DESIGN OF GIBSON DAM.

caused by shear were found to equal or exceed those produced by thrust and moment. Both positive and negative moments were found to be decreased numerically at all points; the thrust was increased materially at all points; the line of thrust was found to follow more closely the center line of the arch, reducing the eccentricity greatly at both crown and haunch; and the resultant compressive stresses were not changed greatly while the tensile stresses were decreased appreciably. Fig. 50 illustrates these effects as calculated for a comparatively short, thick, symmetrical arch, under a uniform load, investigated in connection with the design of another dam.

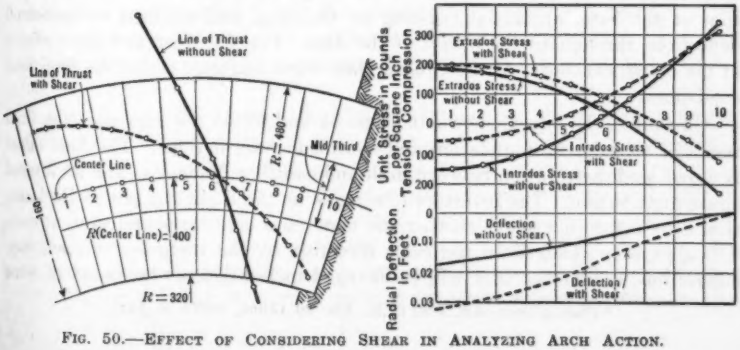


FIG. 50.—EFFECT OF CONSIDERING SHEAR IN ANALYZING ARCH ACTION.

Consideration of the shear effect in analyzing a symmetrical arch by the methods outlined in Table 5, requires the inclusion of columns for the following quantities, the notation being the same as that in Table 5:

$$V_L = \text{the shear due to external loads.}$$

$$\frac{3}{E}\sum\frac{S\,V_L\sin\alpha}{T} = \text{the } x\text{-component of deflection due to } V_L.$$

$$H_c\sin\alpha = \text{the shear component of the crown thrust.}$$

$$H_c\sin\alpha - V_L = \text{the net shear, } V.$$

$$3\,\frac{V\,s}{E\,T} = \text{the shear deformation.}$$

$$\sum\frac{3\,V\,s\cos\alpha}{E\,T} = \text{the } y\text{-component of deflection due to } V.$$

$$-\sum\frac{3\,V\,s\sin\alpha}{E\,T} = \text{the } x\text{-component of deflection due to } V.$$

The inclusion of the effect of abutment and foundation deformation in the cantilever and arch calculations was based on the formulas developed by Fredrik Vogt, Assoc. M. Am. Soc. C. E.* For the longer, thinner arches near the top of the dam, the inclusion of the abutment deformation effect did not result in material changes. When such arches were analyzed under the same water load the deflections were increased slightly, but the moments, thrusts, eccentricities, and stresses were not appreciably altered. However, as regards comparatively short, thick arches the effects were important. For such arches the inclusion of the abutment deformation effect increased the positive moments, decreased the negative moments, decreased the thrust, increased both compressive and tensile stresses at the crown, decreased both compressive and tensile stresses at the abutment, increased the eccentricity of the line of thrust, and practically doubled the deflections. Fig. 51 shows the results of an analysis of one of the heavier arches of the 405-ft. Owyhee Dam now being built on the Owyhee Irrigation Project of Eastern Oregon, no direct comparisons being available for the arches of the Gibson Dam.

In the analysis of cantilevers under the same loads consideration of foundation deformations resulted in increased bending, but did not alter the stresses. Consideration of foundation movements introduced an initial shear deflection at the base, an initial twisting at the base, and resulted in increased deflections throughout the height of the dam. Fig. 52 illustrates these effects at the crown cantilever of the Gibson Dam when analyzed under the final load conditions.

Of course, the effects shown in Figs. 51 and 52 do not represent the final result of considering foundation and abutment deformations. The final effect is a new load distribution between arch and cantilever elements and an altered stress distribution. The investigations made by the U. S. Bureau of Reclamation thus far do not show whether the maximum cantilever and arch stresses will always be changed in the same direction by the inclusion of such considerations, or whether they will be changed in location, or increased in some

* *Transactions*, Am. Soc. C. E., Vol. 90 (June, 1927), p. 554.

places and decreased in others. A great many uncertainties are involved in the consideration of foundation and abutment movements, such as the value of the modulus of elasticity for the rock formations. In the case of the Gibson Dam the modulus of elasticity for the limestone foundation and abutments was assumed to be the same as for the concrete which, on the basis of laboratory tests, was taken as 2 000 000 lb. per sq. in. As Dr. Vogt stated in the previously mentioned discussion, "no calculation of this kind may claim to be more than a rough approximation."

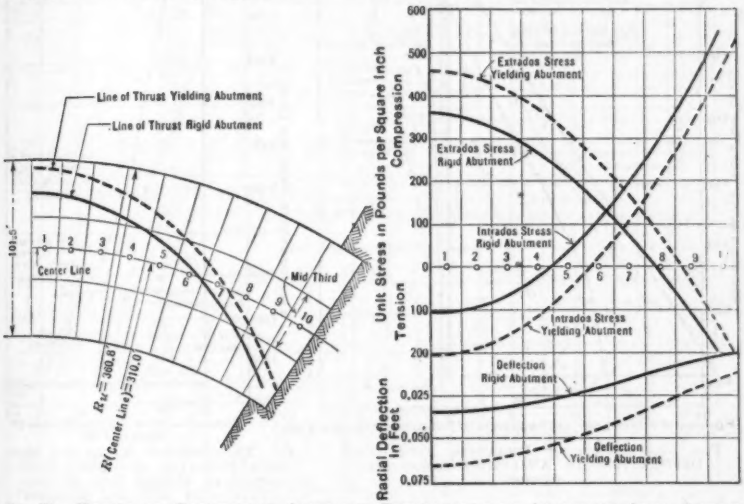


FIG. 51.—EFFECT OF CONSIDERING ABUTMENT DEFORMATION IN ANALYZING ARCH ACTION, OWYHEE DAM.

Assuming arch action to begin as soon as the reservoir begins to fill simply means that the arch deflections are to be measured from the up-stream cantilever deflections which occur when the reservoir is empty, instead of from a vertical axis as was done in the original calculations. Since the contraction joints are to be thoroughly grouted during the late winter, after the setting heat has been dissipated, the assumption made in the final analysis is believed to be more nearly correct than the one originally made. A number of electrical resistance thermometers are being installed in the Gibson Dam so that definite knowledge of internal temperatures will be available at the time the joints are grouted.

Figs. 53 and 54 compare the results obtained by the revised methods of analysis with those obtained by the original methods. Fig. 53 shows the differences in tension areas and maximum cantilever stresses at the crown cantilever, while Fig. 54 shows the difference in load distribution and deflections at the cantilevers, *A*, *C*, *F*, *G*, *H*, and *I*. By referring to Fig. 53 it will be noticed that for the crown cantilever the final result of all the revisions in methods was a slight reduction in tension area and maximum stresses, the latter being reduced to stresses parallel to the down-stream face by Cain's

theory in both cases. Reference to Fig. 54 shows that the deflection and load distribution curves were not altered appreciably, except in the case of the latter, at the lower elevations of the more central cantilevers where appreciable loads were transferred from the cantilever elements to the arch elements.

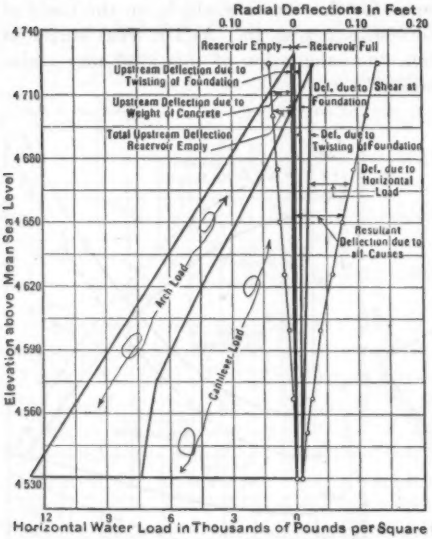


FIG. 52.—EFFECT OF INCLUDING FOUNDATION DEFORMATION IN ANALYZING CANTILEVER ACTION.

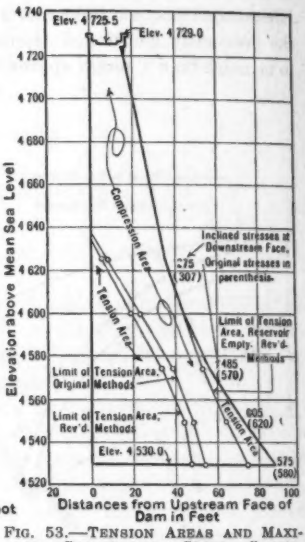


FIG. 53.—TENSION AREAS AND MAXIMUM STRESSES IN CROWN CANTILEVER OF GIBSON DAM BY ORIGINAL AND REVISED METHODS OF CALCULATION.

Fig. 55 shows the maximum cantilever and arch stresses in the Gibson Dam as calculated by the revised methods of analysis. Similar data are available for the original investigations, but the stresses are in no cases greatly different from the final results. The several revisions introduced since the first analysis was made, decreased the maximum cantilever stress from 620 to 605 lb. per sq. in., but did not change its original location which had been found to be at Elevation 4 560 on the down-stream face of the dam, as shown in Fig. 53. They increased the maximum arch stress from 309 to 364 lb. per sq. in. and changed its location from the extrados at Elevation 4 675 at the crown to the extrados at Elevation 4 650 at the crown. Maximum arch stresses at the lower elevations were increased by greater proportional amounts, due to the change in load distribution, but the final stresses were lower than at Elevation 4 650, as shown in Fig. 55.

The results of the revised analysis of the Gibson Dam were very gratifying. They showed that the section as originally designed is satisfactory and that the maximum stresses to be expected will be well within the limits of safety. The fact that the revised analysis did not show results greatly different from the original analysis does not mean that the modifications were not worth while. It simply means that, in the case of the Gibson Dam, the effects

of the various changes tended to compensate, so that the resultant effect was not of serious magnitude. For dams of different cross-section, in canyons of different shape, the final result of such changes might not be so satisfactory.

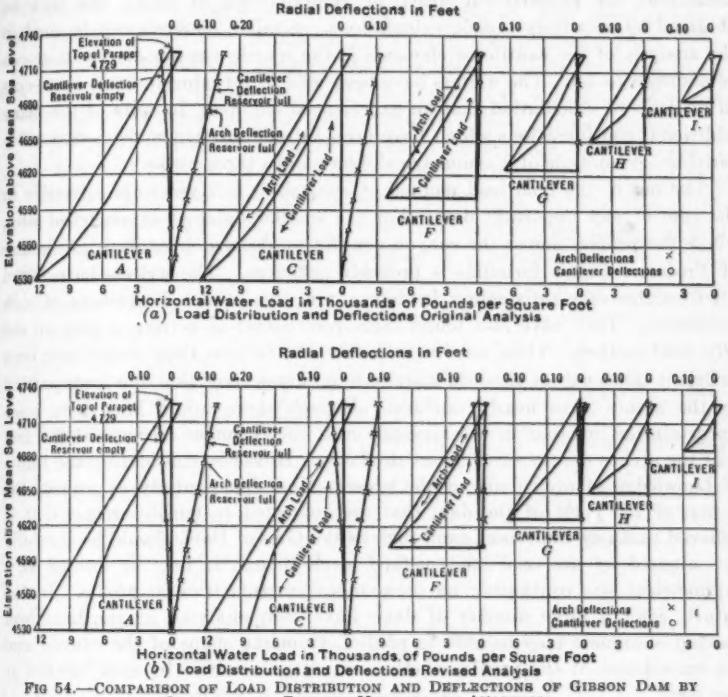


FIG 54.—COMPARISON OF LOAD DISTRIBUTION AND DEFLECTIONS OF GIBSON DAM BY ORIGINAL AND REVISED METHODS OF ANALYSIS.

The use of the trial load method of analysis is not so laborious as might first be supposed, especially after one becomes familiar with the action of arch dams and the changes in arch and cantilever deflections to be expected from

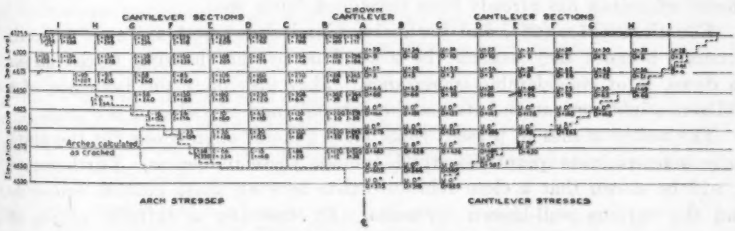


FIG. 55.—MAXIMUM CANTILEVER AND ARCH STRESSES IN THE GIBSON DAM CALCULATED BY REVISED METHODS OF ANALYSIS.

certain changes in load application. Of course, the use of a calculating machine is indispensable in such an investigation. The computation of unit load deflections for the arches results in a considerable saving of time since the deflections for proportional loads, or combinations of loads, can then be obtained by relatively simple calculations. Similar methods can be used in the analysis of the cantilever elements if the tensile stresses are small or can be assumed to act. The writers have been able to introduce enough short-cuts in methods of calculation that an experienced engineer, in spite of the many additional considerations which have been introduced, can make a reasonably satisfactory analysis of a symmetrical dam within three weeks.

The use of the trial load method of analysis is believed to be advisable in the case of any important dam. For the smaller dams at symmetrical sites, where the profiles across the canyon contain no decided irregularities, the use of Professor Cain's formulas is probably sufficient. The writers have found his formulas very satisfactory in determining the proper dimensions of such structures. They have also found them very useful in getting a start on the trial load method. Their use in the case of the Gibson Dam might have been sufficient inasmuch as the load distribution curves show that the loads carried by the arches were nearly uniform, although there would have been some uncertainties involved in the application of the formulas because of the fact that the arches are not of constant thickness. However, considering the limits of knowledge of arch dams at the present time (particularly as regards the action of the parts of the dam that are subjected to tensile stresses), it is believed that any important dam, such as the Gibson Dam, should be carefully investigated by the trial load method, even though it may be located at a symmetrical site containing no pronounced irregularities in profile. In the future, after a large number of dams have been analyzed by the trial load method, designers may be able to predict, from the shape of the canyon and the smoothness of the profile, whether or not an analysis by such method is necessary.

A. Floris,* Esq. (by letter).—The analysis of arches and particularly of arch dams has been treated repeatedly in recent years. Various writers have proposed formulas for the purpose of improving those in current use. This paper is another attempt in this direction in spite of the fact that the theory of arches has already been developed fairly well.

Equations (27), (28), and (29), also found elsewhere,† are not "perfectly general" because they neglect the work done by the shearing forces which, in dams, is of considerable importance. They also omit the influence of a difference in temperature between the extrados and intrados of the arch.

The writer wishes to present some more general expressions for the statically indeterminate quantities which include these influences. Furthermore, it will be shown that a close relation exists between these general equations and the various well-known formulas. By choosing a suitable origin of

* Los Angeles, Calif.

† "Plain and Reinforced Concrete Arches," by J. Melan; translated by D. B. Steinman, M. Am. Soc. C. E., N. Y., 1915, p. 30.

co-ordinates to which the fundamental equations are referred and by successive simplifications, these special formulas can be easily derived.

It is well known that a curved prismatic bar restrained at its supports is a statically indeterminate system in the third degree. For these unknown quantities, therefore, three equations of deformation are necessary. There are a number of methods of analysis available which provide these equations. The theorem of the derivatives of the internal work will be used herein because it leads to the solution of the problem in a most simple manner.

Generally, the theory of structures is based on assumptions* in order to make a practical treatment of the subject possible. With this object in view the writer has made the usual assumptions that Hooke's law and Bernoulli's hypothesis that plane sections remain plane after bending, are valid and that the shearing stresses are distributed according to a parabolic law.

If the radius of curvature of a curved bar is not less than about five times its thickness† the general equation of the internal work with all possible effects included is:‡

$$u = \int \frac{M^2\, d\,s}{2\,E\,I} + \int \frac{N^2\, d\,s}{2\,E\,A} + \kappa \int \frac{Q^2\, d\,s}{2\,G\,A} + \int \varepsilon \frac{\Delta\, t}{h}\, M\, d\,s + \int \varepsilon\, t\, N\, d\,s \ldots (159)$$

Differentiating Equation (159) with respect to X, Castigliano's theorem of partial derivatives is obtained:

$$L = \frac{\delta\, u}{\delta\, X} = \int \frac{M}{E\,I} \frac{\delta\, M}{\delta\, X} d\,s + \int \frac{N}{E\,A} \frac{\delta\, N}{\delta\, X} d\,s + \kappa \int \frac{Q}{G\,A} \frac{\delta\, Q}{\delta\, X} d\,s + \int \varepsilon \frac{\Delta\, t}{h} \frac{\delta\, M}{\delta\, X} d\,s + \int \varepsilon\, t \frac{\delta\, N}{\delta\, X} d\,s \ldots\ldots\ldots\ldots\ldots (160)$$

If the supports are rigid, this expression will be:

$$\int \frac{M}{E\,I} \frac{\delta\, M}{\delta\, X} d\,s + \int \frac{N}{E\,A} \frac{\delta\, N}{\delta\, X} d\,s + \kappa \int \frac{Q}{G\,A} \frac{\delta\, Q}{\delta\, X} d\,s + \int \varepsilon \frac{\Delta\, t}{h} \frac{\delta\, M}{\delta\, X} d\,s + \int \varepsilon\, t \frac{\delta\, N}{\delta\, X} d\,s = 0 \ldots\ldots\ldots\ldots\ldots\ldots (161)$$

which is the theorem of least work.

By using these equations, any prismatic bar can be analyzed in the shortest and most elegant manner. On this fact is based the superiority of the theorem of internal work as compared with the cumbersome and lengthy method preferred by the authors.

The integrals in Equations (159), (160), and (161) are those due to bending moments, normal forces, shearing forces, unequal temperature variation within the section of the bar, and uniform temperature change throughout the section, respectively.

* "Calcul des arcs élastiques," par S. Timoshenko, Paris, 1922, pp. 4–6.

† "Applied Elasticity," by S. Timoshenko and J. M. Lessels, East Pittsburgh, 1925, p. 221; or "Die neueren Methoden der Festigkeitslehre und der Statik der Baukonstruktionen," von H. Müller-Breslau, Berlin, 1924, p. 242.

‡ "Kinetic Theory of Engineering Structures," by D. A. Molitor, M. Am. Soc. C. E., N. Y., 1911, p. 42.

In these equations, $G = E \frac{m}{2(m+1)}$*, $\nu = 2\kappa \frac{m+1}{m}$, and for rectangular sections, $\kappa = \frac{6}{5}$†.

Substituting these values, the integral of the shearing forces in Equations (159), (160), and (161) becomes,

$$\nu \int \frac{Q}{E A} \frac{\delta Q}{\delta x} d s$$

Concrete tests show‡ that m varies between 6.5 and 8.0. Taking an average value of 7.25, then $\nu = 2.74$ and not 3.0 or 2.88 as is assumed by various writers.

A restrained arch which is either unsymmetrically formed or unsymmetrically loaded is statically indeterminate in the third degree. Taking any point as the origin of the co-ordinates and introducing X, Y, and μ as the statically indeterminate quantities, the bending moment, normal force, and shearing force for any point on the arch axis will be (see Fig. 56):

$$M = M_0 + X y + Y x + \mu \ldots\ldots\ldots\ldots\ldots\ldots (162)$$

$$N = N_0 + X \cos \phi + Y \sin \phi \ldots\ldots\ldots\ldots\ldots\ldots (163)$$

$$Q = Q_0 - X \sin \phi + Y \cos \phi \ldots\ldots\ldots\ldots\ldots\ldots (164)$$

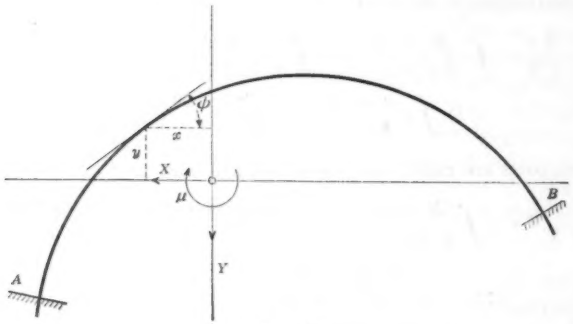


FIG. 56.

The corresponding derivatives will be:

$$\frac{\delta M}{\delta X} = y; \quad \frac{\delta M}{\delta Y} = x; \quad \frac{\delta M}{\delta \mu} = 1 \ldots\ldots\ldots (165)$$

$$\frac{\delta N}{\delta X} = \cos \phi; \quad \frac{\delta N}{\delta Y} = \sin \phi; \quad \frac{\delta N}{\delta \mu} = 0 \ldots\ldots\ldots (166)$$

$$\frac{\delta Q}{\delta X} = -\sin \phi; \quad \frac{\delta Q}{\delta Y} = \cos \phi; \quad \frac{\delta Q}{\delta \mu} = 0 \ldots\ldots\ldots (167)$$

* "Strength of Materials," by G. F. Swain, Past-President, Am. Soc. C. E., N. Y., 1924, p. 62.

† "Théorie de l'équilibre des systèmes élastiques et ses applications," Turin, 1879. English translation by E. S. Andrews, Lond., 1919, p. 226.

‡ "Researches in Concrete," by W. K. Hatt, M. Am. Soc. C. E., *Bulletin No. 24*, Eng. Experiment Station, Purdue Univ., p. 38.

M_0, N_0, and Q_0, in Equations (162) to (164) are the bending moment, normal force, and shearing force, respectively, of the statically determinate system which is sometimes called the principal or base system.

Because there are three unknowns, X, Y, and μ, Equation (161) must be written out three times as follows:

$$\int_A^B \frac{M}{EI}\frac{\delta M}{\delta X}ds + \int_A^B \frac{N}{EA}\frac{\delta N}{\delta X}ds + \nu\int_A^B \frac{Q}{EA}\frac{\delta Q}{\delta X}ds + \int_A^B \varepsilon\frac{\Delta t}{h}\frac{\delta M}{\delta X}ds + \int_A^B \varepsilon t\frac{\delta N}{\delta X}ds = 0. \quad (168)$$

$$\int_A^B \frac{M}{EI}\frac{\delta M}{\delta Y}ds + \int_A^B \frac{N}{EA}\frac{\delta N}{\delta Y}ds + \nu\int_A^B \frac{Q}{EA}\frac{\delta Q}{\delta Y}ds + \int_A^B \varepsilon\frac{\Delta t}{h}\frac{\delta M}{\delta Y}ds + \int_A^B \varepsilon t\frac{\delta N}{\delta Y}ds = 0 \quad (169)$$

$$\int_A^B \frac{M}{EI}\frac{\delta M}{\delta \mu}ds + \int_A^B \frac{N}{EA}\frac{\delta N}{\delta \mu}ds + \nu\int_A^B \frac{Q}{EA}\frac{\delta Q}{\delta \mu}ds + \int_A^B \varepsilon\frac{\Delta t}{h}\frac{\delta N}{\delta \mu}ds + \int_A^B \varepsilon t\frac{\delta N}{\delta \mu}ds = 0 \quad (170)$$

In Equations (168), (169), and (170), it is assumed that the uniform temperature variation in the arch is acting in the same direction with the outer forces. This is the case when a drop in temperature takes place in the arch. For a rise in temperature the sign before the integral, expressing this uniform change, must be reversed. Introducing the values given by Equations (162) to (167) in the proper place in Equations (168), (169), and (170), the following expressions are derived:

$$X\left[\int_A^B \frac{y^2}{I}ds + \int_A^B \frac{\cos^2\phi}{I}ds + \nu\int_A^B \frac{\sin^2\phi}{I}ds\right] + Y\left[\int_A^B \frac{xy}{I}ds + (1-\nu)\int_A^B \frac{\sin\phi\cos\phi}{I}ds\right] + \mu\int_A^B \frac{y}{I}ds + \int_A^B \frac{M_0}{I}y\,ds + \int_A^B \frac{N_0}{A}\cos\phi\,ds - \nu\int_A^B \frac{Q_0}{A}\sin\phi\,ds + E\varepsilon\int_A^B \frac{\Delta t}{h}y\,ds + E\varepsilon t\int_A^B \cos\phi\,ds = 0 \quad (171)$$

$$X\left[\int_A^B \frac{yx}{I}ds + (1-\nu)\int_A^B \frac{\sin\phi\cos\phi}{A}ds\right] + Y\left[\int_A^B \frac{x^2}{I}ds + \int_A^B \frac{\sin^2\phi}{A}ds + \nu\int_A^B \frac{\cos^2\phi}{A}ds\right] + \mu\int_A^B \frac{x}{I}ds + \int_A^B \frac{M_0}{I}x\,ds + \int_A^B \frac{N_0}{A}\sin\phi\,ds + \nu\int_A^B \frac{Q_0}{A}\cos\phi\,ds + E\varepsilon\int_A^B \frac{\Delta t}{h}x\,ds + E\varepsilon t\int_A^B \sin\phi\,ds = 0 \quad (172)$$

$$X\int_A^B \frac{y}{I}ds + Y\int_A^B \frac{x}{I}ds + \mu\int_A^B \frac{ds}{I} + \int_A^B \frac{M_0}{I}ds + E\varepsilon\int_A^B \frac{\Delta t}{h}ds = 0 \quad (173)$$

Equations (171), (172), and (173) are the most general expressions for the statically indeterminate quantities of an unsymmetrically formed and un-

symmetrically loaded arch with complete restraint at its abutments. However, if the abutments are not perfectly rigid then, instead of being equal to zero, Equations (168), (169), and (170) must be made equal to these elastic movements; or, what is the same thing, Equation (160) must be used. Obviously, the expressions for these movements would appear in Equations (171), (172), and (173), which can be solved only if the magnitudes of the displacements or the rotations are measured or are known in advance in some way. For this reason these formulas are omitted because, in practice, the abutments of arch dams are generally assumed to be perfectly rigid.

In order to integrate Equations (171), (172), and (173), the axis of the arch must be shaped according to a mathematical curve and the thickness of the arch must vary according to some known law. Otherwise, the desired result must be obtained by summation as is done by the authors.

Neglecting the influence of the shearing forces and of the unequal temperature variation on the arch and also referring the system to the origin of the co-ordinates at the crown where the arch is assumed to be cut in two, Equations (171), (172), and (173) immediately become:

$$X\left[\int_A^B \frac{y^2}{I}\,ds+\int_A^B \frac{\cos^2\phi}{A}\,ds\right]+Y\left[\int_A^B \frac{x\,y}{I}\,ds+\int_A^B \frac{\sin\phi\cos\phi}{A}\,ds\right]$$
$$+\mu\int_A^B \frac{y}{I}\,ds+\int_A^B \frac{M_0}{I}y\,ds+\int_A^B \frac{N_0}{A}\cos\phi\,ds+E\varepsilon t\int_A^B \cos\phi\,ds=0\ldots(174)$$

$$X\left[\int_A^B \frac{y\,x}{I}\,ds+\int_A^B \frac{\cos\phi\sin\phi}{A}\,ds\right]+Y\left[\int_A^B \frac{x^2}{I}\,ds+\int_A^B \frac{\sin^2\phi}{A}\,ds\right]$$
$$+\mu\int_A^B \frac{x}{I}\,ds+\int_A^B \frac{M_0}{I}x\,ds+\int_A^B \frac{N_0}{A}\sin\phi\,ds+E\varepsilon t\int_A^B \sin\phi\,ds=0\ldots(175)$$

$$X\int_A^B \frac{y}{I}\,ds+Y\int_A^B \frac{y}{I}\,ds+\mu\int_A^B \frac{ds}{I}+\int_A^B \frac{M_0}{I}\,ds=0\ldots(176)$$

which are the authors' Equations (29), (28), and (27), in a somewhat different form.

In these expressions the integration must be extended over the entire arch. However, bearing in mind that in Equations (162) and (163) the terms containing Y must be negative for the right half of the arch, the integrals in Equation (175) are obtained by subtracting the integrals of the right half from those of the left half. On the other hand, x^2 and $\sin^2\phi$ being positive, the corresponding integrals must be added and not subtracted. In the same way the integrals of Y in Equations (174) and (176) must be also subtracted. The remaining integrals are the sums of the left and right-hand parts of the arch.

Considering, furthermore, that the integrals, $\int_A^B \cos\phi\,ds$ and $\int_A^B \sin\phi\,ds$, are the projections of the arch upon the x and y-axes, respectively, it follows therefrom that they define the distances between the projections on the x and y-axes of the points, A and B, of the arch at its abutments, respectively. With this fact in view the term denoting the influence of the uniform temperature variation in the authors' Equation (29) seems to be incorrect.

In a symmetrical arch, but with an unsymmetrical loading, the integrals corresponding to the left and right-hand parts of the arch are equal, so that their difference will be zero.

In this case, therefore, the effect will be to make the integrals of Y in Equations (174) and (176) and the integrals of X and μ in Equation (175) equal to zero. At the same time the last integral in Equation (175) vanishes also for the reason that the deformation of the arch due to a uniform temperature variation being symmetrical no Y-force can be produced. This result is based on the supposition that the arch is cut in two at the crown.

This choice of the principal system facilitates the computations appreciably because, by taking the origin of the co-ordinates at the left abutment, for instance, and cutting the arch at this point, Equations (174), (175), and (176) must be used even in a symmetrical arch under unsymmetrical loading. It is needless to say that in this case the integrals in these equations are the sums and not the differences of the two halves of the arch.

In bridges the thickness of the arch is small as compared with the radius and, consequently, the work done by the shearing forces is always neglected. The deformation due to the normal forces is also neglected if the arch is not too flat. In Equations (168), (169), and (170), therefore, the integrals due to the bending moments alone are taken into consideration and the general Equations (171), (172), and (173), neglecting the influence of temperature, are simplified as follows: In Equation (171), the second, third, fifth, eighth, ninth, tenth, and eleventh integrals vanish; in Equation (172), the second, fourth, fifth, eighth, ninth, tenth, and eleventh integrals are equal to zero; and in Equation (173), the last integral disappears.

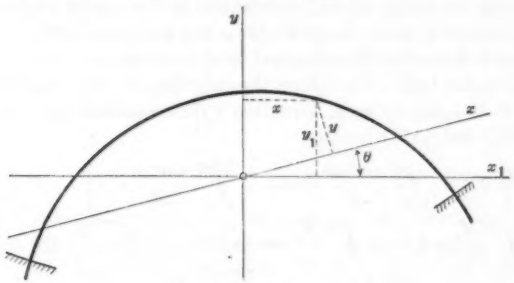


FIG. 57.

Müller-Breslau was the first to show* that the integrals, $\int_A^B \frac{y}{I}\,ds$ and $\int_A^B \frac{x}{I}\,ds$, representing the static moments, and $\int_A^B \frac{x\,y}{I}\,ds$, the centrifugal moment of the symmetrical arch ring, vanish if they are referred to the elastic center of gravity of the ring. The elastic center, on the other hand, is determined by considering $\frac{ds}{I}$ as forces acting in two directions. In the case of a symmetrical arch the y-axis

* "Vereinfachung der Theorie der statisch unbestimmten Bogentraeger," H. von Müller-Breslau, *Zeitschrift* des Architekten und Ingenieur-Vereins zu Hannover, 1884, p. 575.

being an axis of symmetry the condition, $\int_A^B \frac{x}{I}\, d\,s = 0$, is satisfied without further proof. However, if the arch is unsymmetrical, the centrifugal moment, $\int_A^B \frac{x\,y}{I}\, d\,s$, vanishes only in the case when it is referred to an inclined axis, the slope of which is given by* (see Fig. 57):

$$tg\,\theta = \frac{\int_A^B \frac{x\,y_1}{I}\, d\,s}{\int_A^B \frac{x^2}{I}\, d\,s}$$

The great advantage of this arrangement is that all the more complicated formulas are reduced to three single equations with one unknown each. They are obtained as follows: In Equation (171) all integrals except the first and seventh are equal to zero; in Equation (172) all integrals vanish with the exception of the third and seventh; and, lastly, in Equation (173) all integrals, except the third and fourth, are equal to zero.

In arch dams, in which the work done by the normal forces usually predominates, there are no methods available to reduce Equations (171), (172), and (173) to three equations with one unknown each. It is true that in bridges with a comparatively small rise it can be assumed approximately in Equation (163) that $X \cos \phi = X$ and $Y \sin \phi = 0$, if a special analysis is made for the dead load of the arch.† With the aid of these assumptions it is possible to reduce Equations (171), (172), and (173), to three equations with one unknown each by referring the origin of the co-ordinates to the elastic center of the arch. In arch dams, however, these simplifications are not permissible.

Usually, arch dams are symmetrical and, as such, are always stressed by a symmetrical water load. Therefore, the calculations are simplified considerably. Since Y is equal to zero, Equation (172) vanishes and the remaining Equations (171) and (173) become:

$$X\left[\int_A^B \frac{y^2}{I}\, d\,s + \int_A^B \frac{\cos^2 \phi}{A}\, d\,s + \nu \int_A^B \frac{\sin^2 \phi}{A}\, d\,s\right] + \mu \int_A^B \frac{y}{I}\, d\,s + \int_A^B \frac{M_0}{I}\, y\, d\,s + \int_A^B \frac{N_0}{A} \cos \phi\, d\,s - \nu \int_A^B \frac{Q_0}{A} \sin \phi\, d\,s + E\,\varepsilon \int_A^B \frac{\Delta\, t}{h}\, y\, d\,s + E\,\varepsilon\, t \int_A^B \cos \phi\, d\,s = 0 \quad \ldots\ldots\ldots (177)$$

$$X \int_A^B \frac{y}{I}\, d\,s + \mu \int_A^B \frac{d\,s}{I} + \int_A^B \frac{M_0}{I}\, d\,s + E\,\varepsilon \int_A^B \frac{\Delta\, t}{h}\, d\,s = 0 \quad \ldots\ldots (178)$$

These are the fundamental equations derived by Professor Cain arranged in a more general form.‡ They are referred to the origin of the co-ordinates at the crown.

* "Gewölbte Brücken," von E. Mörsch, *Beton-Kalender*, Berlin, 1915, p. 303.

† *Loc. cit.*, pp. 295 and 302.

‡ "The Circular Arch Under Normal Loads," by William Cain, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), Equations (7) and (9), p. 237.

Assuming the elastic center of the arch as the origin of the co-ordinates, then, $\int_A^B \frac{y}{I}\, d\,s = 0$, as stated previously, and Equations (177) and (178) become:

$$X\left[\int_A^B \frac{y^2}{I}\, d\,s + \int_A^B \frac{\cos^2\phi}{A}\, d\,s + \nu \int_A^B \frac{\sin^2\phi}{A}\, d\,s\right] + \int_A^B \frac{M_0}{I}\, y\, d\,s + \int_A^S \frac{N_0}{A}\cos\phi\, d\,s - \nu \int_A^B \frac{Q_0}{A}\sin\phi\, d\,s + E\,\varepsilon \int_A^B \frac{\Delta\, t}{h}\, y\, d\,s + E\,\varepsilon\, t \int_A^B \cos\phi\, d\,s = 0 \quad \ldots\ldots\ldots\ldots\ldots\ldots (179)$$

$$\mu \int_A^B \frac{d\,s}{I} + \int_A^B \frac{M_0}{I}\, d\,s + E\,\varepsilon \int_A^B \frac{\Delta\, t}{h}\, d\,s = 0 \quad \ldots\ldots\ldots (180)$$

in which the unknowns, X and μ, are separated. Similar expressions, in a less complete form, were developed by Dr. N. Kelen for arches of constant thickness.*

When it is required to determine the influence of an unequal temperature variation on an arch of constant thickness, h, Equation (179) vanishes entirely because,

$$E\,\varepsilon\, \frac{\Delta\, t}{h} \int_A^B y\, d\,s = 0$$

in which,

$$\int_A^B \frac{y}{I}\, d\,s = \int_A^B y\, d\,s = 0$$

since it is referred to the elastic center of the arch. The remaining Equation (180), therefore, will be,

$$\mu \int_A^B \frac{d\,s}{I} + E\,\varepsilon\, \frac{\Delta\, t}{h} \int_A^B d\,s = 0 \quad \ldots\ldots\ldots\ldots\ldots (181)$$

Substituting for I its equivalent, $\frac{h^3}{12}$:

$$\mu = E\,\varepsilon\, \frac{h^2}{12}\, \Delta\, t\dagger \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (182)$$

in which, $\Delta\, t$ is the difference, $t_1 - t_2$, between the up-stream and down-stream temperatures. This difference determines the sign of μ since compression is developed in the warmer side of the arch.

One can introduce a further simplification by designing the arch so that the line of resistance in the principal system coincides exactly with the arch axis. In this case $\int_A^B \frac{M_0}{I}\, y\, d\,s$ and $\int_A^B \frac{M_0}{I}\, d\,s$ in Equations (179) and (180) vanish; consequently, $\int_A^B \frac{Q_0}{I} \sin\phi\, d\,s$ is equal to zero, since the shearing forces are produced by the difference of the moments in two adjacent sections of the arch which presumably are equal to zero. Equations (179) and (180), therefore, take the extremely simple form:

* "Die Staumauern," von N. Kelen, Berlin, 1926, p. 17.

† "Statica delle dighe per laghi artificiali," C. Guidi, Turin, 1921, p. 49.

$$X\left[\int_A^B \frac{y^2}{I}\,ds + \int_A^B \frac{\cos^2\phi}{A}\,ds + \nu\int_A^B \frac{\sin^2\phi}{A}\,ds\right] + \int_A^B \frac{N_0}{A}\cos\phi\,ds + E\,\varepsilon\int_A^B \frac{\Delta t}{h}\,y\,ds + E\,\varepsilon\,t\int_A^B \cos\phi\,ds = 0 \quad \ldots(183)$$

which is Professor E. Moersch's generalized equation.*

It is unfortunate that this elegant and simple method of analysis (Equation (183)) has found little favor among American designers although it was made known several years ago.†

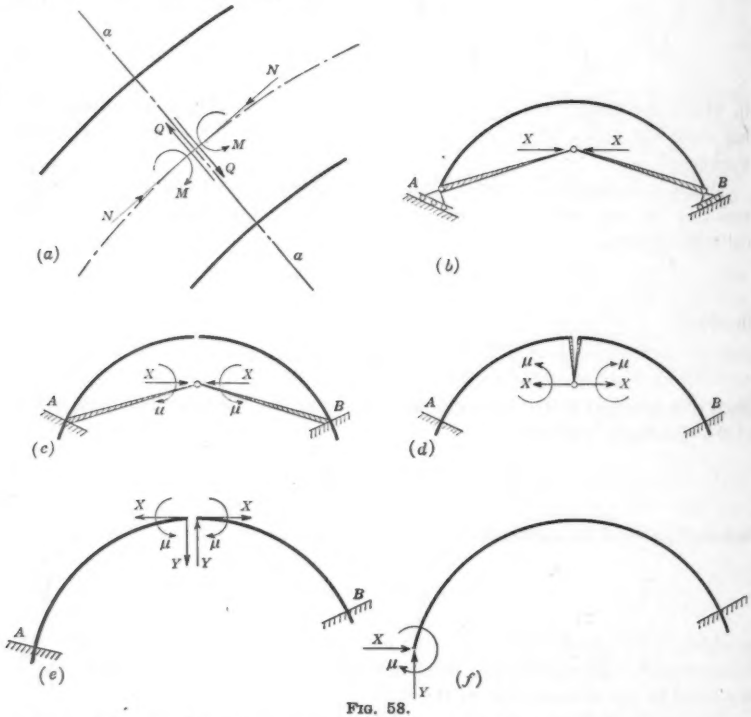


FIG. 58.

Since Equations (177) to (181), inclusive, and Equation (183) are applied to symmetrically formed and symmetrically loaded arches, it follows that their integration can be referred to one-half of the arch only.

In order to apply correctly the equations already derived to the various principal systems it must be remembered that the positive directions of N, Q, and M to the left and to the right of a given section, a-a, are as shown in Fig. 58(a). It is also assumed that a positive bending moment, M, produces compression in the extrados of the arch.

* "Berechnung kreisförmiger Gewölbe gegen Wasserdruck," von E. Mörsch, *Schweizerische Bauzeitung*, 1908, p. 233.

† "The Relation Between Deflections and Stresses in Arch Dams," by F. A. Noetzli, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 302.

With these definitions in mind the statically indeterminate quantities, X, Y, and μ, in Fig. 58(b), 58(c), and 58(f), being reactions, are acting to the left of a given section, a-a. On the other hand, the same quantities in Fig. 58(d) and 58(e) belong to the arch itself and, consequently, they must act to the left side of this section.

In both cases, Fig. 58(c) and 58(d), the moment due to X is, for sections below the x-axis, positive, and, for sections above this axis, negative.

The bending moment, M_0, of the curved cantilever, Fig. 58(c), 58(d), 58(e), and 58(f), is negative because it produces tension in the extrados of the arch. On the other hand, the principal system, Fig. 58(b), is free of bending moments, M_0.

The force, X, acting in the elastic center of the arch is sometimes called a pool,* which is misleading from the point of view of precise thinking. This force is a compression, except in the case of a rise in temperature in the arch, and its relative direction depends only on the choice of the principal system.

As regards the most suitable principal system for a given case the following may be of interest.

There are many methods of obtaining this principal system, depending entirely on the ease of making the computations. In a symmetrical arch, the principal system represented in Fig. 58(b) is convenient for uniformly distributed water load and also uniform change of temperature or shrinkage, while either of the principal systems Fig. 58(c) and 58(d) are preferable for the dead load of the arch or for a variable water pressure.† In other words, the first case is applicable to arch dams while the second case is to be used in multiple-arch dams. The unequal temperature variation between the extrados and intrados of the arch belongs to either of the cases shown in Fig. 58(b), 58(c), or 58(d).

If the arch is circular and of constant thickness and the water pressure is uniform, the line of resistance in the principal system coincides with its geometrical axis. In this case Equation (183) is strictly correct. For a variable water pressure and for the dead load of an arbitrarily formed arch, the line of resistance does not coincide with its axis and, consequently, Equations (179) and (180) must be used. However, Equation (183) can be applied also with a sufficient degree of accuracy if the condition that $M_0 = 0$ is not strictly satisfied.‡

In an unsymmetrical arch under water pressure there are no advantages in choosing the elastic center of the arch as the origin of the co-ordinates. For this reason it can be assumed that this origin is either at the crown or at the left abutment. In the first case the principal system consists of two cantilevers, Fig. 58(e), while in the second case one cantilever stretches over the entire span of the arch, Fig. 58(f). To both cases Equations (174), (175), and (176) must be applied.

* "The Design and Construction of Dams," by E. Wegmann, M. Am. Soc. C. E., N. Y., 1927, p. 445.

† "Die Staumauern," von N. Kelen, p. 20.

‡ "Improved Type of Multiple-Arch Dam," by F. A. Noetzli, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXXVII (1924), p. 361.

C. H. HOWELL,* M. AM. SOC. C. E. (by letter).—The interest shown in this paper is appreciated. Mr. Jakobsen states "the authors assume that the arch dam is a continuous structure, not divided by contraction joints or cracks" and "to insure freedom from shrinkage cracks, the authors state that openings will be left in the dam and that these will be filled in the coldest weather."

The first statement is not quite correct. One of the fundamental concepts of the trial load method is the assumption of joints both horizontal and vertical. These joints are considered incapable of transmitting tensile stresses. In this respect this method of analysis differs from others. The writers did not state that closing plugs would be used to "insure freedom" from shrinkage cracks. The statement in the paper is "to compensate, to some degree, for the shrinkage of the concrete." The extent of compensation depends on the temperature of the dam at the time of closure, the age of the previously poured concrete, and the number of plugs.

The writers had no idea of suggesting that the plugs would completely offset shrinkage. Mr. Jakobsen states that "in time, when more is known about shrinkage, it can be (and, of course, should be) included in the calculations". This sentence expresses exactly the thought of the writers.

Mr. Jakobsen calls attention to the fact that on diagrams in Figs. 5 and 8 no load is shown on the arches at the abutments. He properly questions the correctness of this, as does Professor Cain. Fig. 59 shows final load diagrams for two Horse Mesa arches. It is apparent that, at the abutments, the arch loads vary within wide limits, as stated by Mr. Jakobsen and Professor Cain. Figs. 5 and 8 are for the earliest studies; Fig. 59 was omitted from the original paper.

The method of determining the proportional distribution at the abutments suggested by Mr. Jakobsen, is not quite general. For example, let Fig. 60 (*a*) represent a part of the profile where a sudden change in slope occurs. Consider three equal loads, p_1, p_2 and p_3. Using Mr. Jakobsen's notation and method, the distribution of shear stress for p_1 (see Fig. 60 (*b*)) is expressed by,

$$\frac{p_1 - c}{c} = \frac{\sin 30°}{\cos 30°} = \frac{0.5000}{0.866} = 0.577 \ldots\ldots\ldots\ldots (184)$$

for p_2 (see Fig. 60 (*c*)),

$$\frac{p_2 - c}{c} = \frac{\sin 30°}{\cos 60°} = \frac{0.500}{0.500} = 1.00 \ldots\ldots\ldots\ldots\ldots (185)$$

and for p_3 (see Fig. 60 (*d*)),

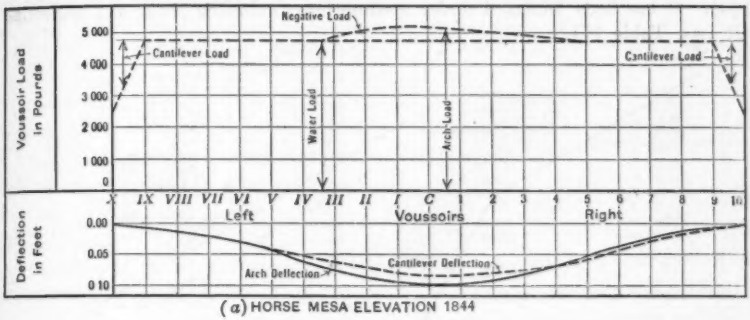
$$\frac{p_3 - c}{c} = \frac{\sin 60°}{\cos 60°} = \frac{0.866}{0.500} = 1.732 \ldots\ldots\ldots\ldots (186)$$

Because they are very close together, the loads must have approximately the same distribution and not the wide variation indicated in Equations (184), (185), and (186).

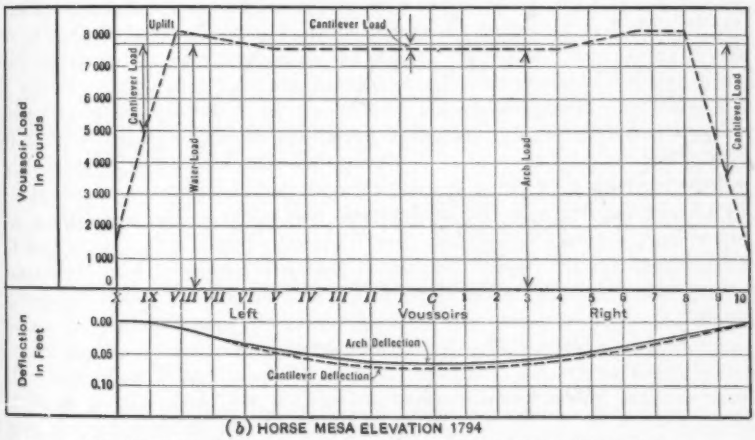
The writer thinks that Mr. Jakobsen misunderstands the application of Fig. 15. In computing the secondary arches, a new center line is drawn through the middle of the uncracked part of the original arch. The new

* Chf. Engr., J. G. White Eng. Corporation, S. en C., City of Mexico, Mexico.

analysis is then made on this new center line. Its length is divided equally and the moments of inertia computed on sections normal to it. No oblique sections are used. The lines in Fig. 15 are intended to show the division of the water load into portions tributary to the divisions of the axis.



(a) HORSE MESA ELEVATION 1844



(b) HORSE MESA ELEVATION 1794

FIG. 59.—FINAL LOAD DIAGRAMS FOR TWO ARCHES, HORSE MESA DAM.

Mr. Rich, as well as others, states that in short thick arches the deflections caused by shear are appreciable and should be considered. When such arches are assumed to be capable of resisting tension the effect of shear is considerable. However, if the area affected by tension is excluded and if the analysis is made on the more flexible interior arch the effect of shear has not been found to be important in such arches investigated by the writer. This has also been the case in regular arches entirely in compression. Shear is the thrust times the sine of the angle between the arch axis and line of thrust. If there is no tension in the arch the angles between the line of thrust and arch axis cannot be large because the latter is within the middle third at all points.

However, it is well to include the effect of shear since it is a factor, although not a serious one, and does not add materially to the work.

The writer has checked Equations (55), (56) and (57) and believes them to be correct. In accordance with Mr. Rich's request, a horizontal section of the arch at Elevation 1 980 of the Calles Dam is shown in Fig. 61 (*a*). It has been analyzed under assumptions including shear and neglecting it (see Table 17).

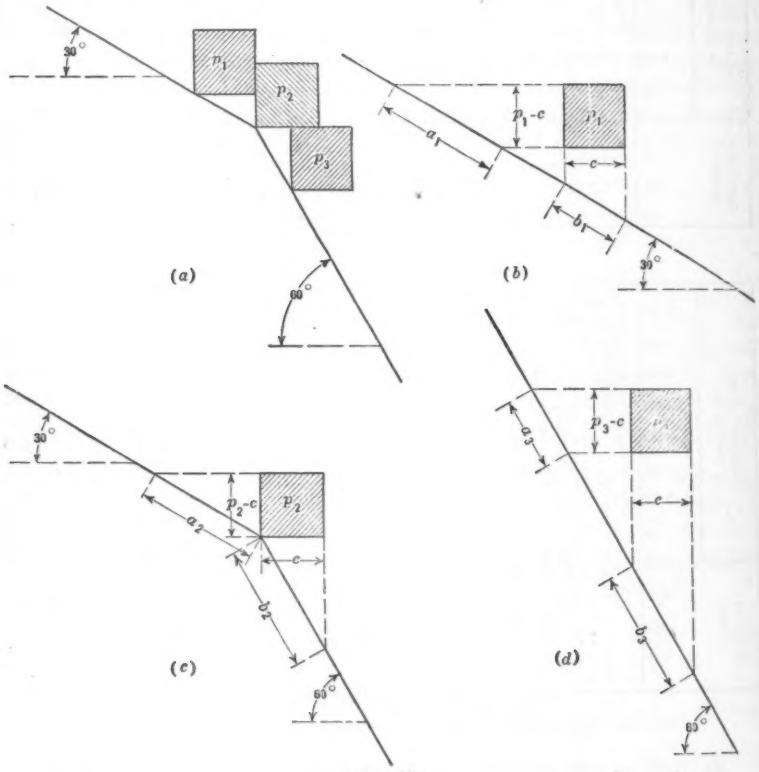


FIG. 60.

The area between the dotted lines and the theoretical extrados (Fig. 61 (*a*)) was found to be in tension in the first analysis. If shear had been considered in connection with tensile stresses the effect would have been considerable because the thrust line was far outside the middle third and formed large angles with the original axis.

Analysis of the "interior arch" (between the theoretical extrados and theoretical intrados) which is entirely in compression, shows that the difference between the positions of the thrust lines with shear and without it is too

small to be platted. The differences in computed deflections and eccentricities as shown in Table 17 are negligible.

TABLE 17.—CALLES DAM AT ELEVATION 1 980.

Joint.	DEFLECTIONS.		ECCENTRICITIES.	
	With shear.	Without shear.	With shear.	Without shear.
Crown	1.121	1.109	0.64010	0.67438
1	1.043	1.078	0.69498	0.72818
2	1.013	1.981	0.77886	0.80861
3	0.899	0.885	0.73775	0.76203
4	0.757	0.742	0.40275	0.42007
5	0.591	0.516	−0.07701	−0.06833
6	0.416	0.423	−0.63963	−0.64102
7	0.248	0.257	−0.82268	−0.83456
8	0.117	0.112	−0.80675	−0.83120
9	0.034	0.032	−0.72409	−0.76259
Springing line	0.000	0.000	−0.73335	−0.78819

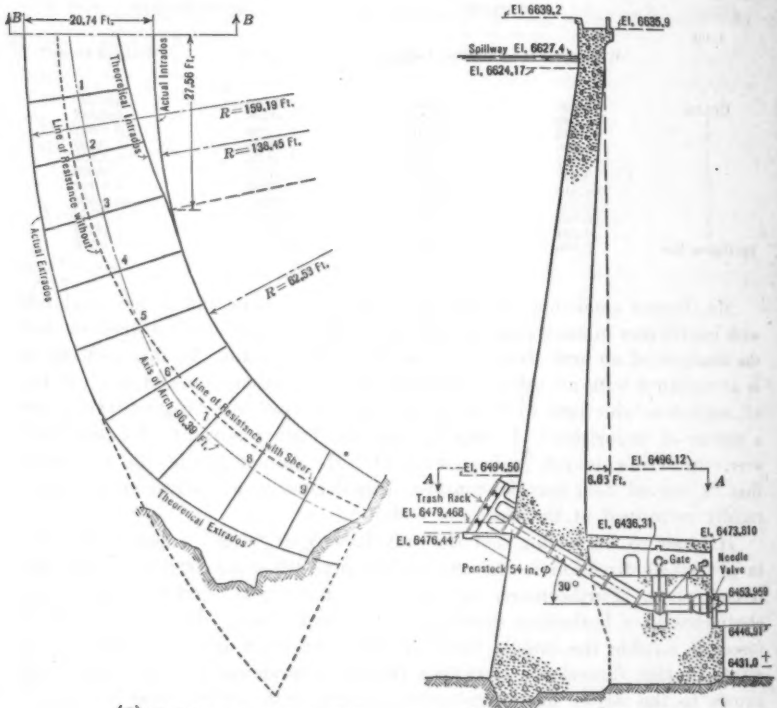
Mr. Turner condemns the conception of arches alone and arches combined with cantilevers in analyzing curved dams. He proposes as the correct method the analysis of an arch dam as if it were a curved plate. In this position he is at variance with all others contributing to the discussion and most, if not all, engineers who have written on the subject. However, "engineering is not a matter of majorities." It may be that Mr. Turner is right. Of this, however, the writer is yet to be convinced. He cannot accept the statement that "a curved dam may be considered analogous to a vertical curved plate rigidly restrained at the base and ends."

It seems to the writer that the only force that "fixes" a dam to its base in a vertical direction is its own weight plus whatever vertical water load it may carry. Furthermore, the only force that "fixes" a curved dam to the abutments in a horizontal direction is the arch thrust. If the resultant of forces is outside the middle third in either direction the dam will tend to rotate in that direction. It cannot then be considered as fixed. All dams known to the writer have construction joints, both vertical and horizontal. The entire base of the dam is a construction joint. The writer knows of no evidence to the effect that tension can be transmitted across ordinary construction joints unless reinforcement is used.

The student in the engineering school mentioned by Mr. Turner would be considered deficient in knowledge of the ordinary theory of concrete design and practice if he proposed to take advantage of the tensile strength of concrete in the design of a jointless concrete beam. Yet some post-graduate engineers with years of experience undertake to analyze unreinforced concrete dams with numerous vertical and horizontal joints as if tensile stresses could be transmitted across them.

Mr. Turner gives two diagrams, Figs. 18 and 20 for deflections of the Stevenson Creek Dam and states, "the graphic method determines relative, not actual deflections" and "the agreement between the relative theoretical and measured deflections [of Engineering Foundation] is sufficiently close to carry conviction."

A diagram of computed deflections which is merely similar in appearance to a diagram of observed deflections, but which is entirely lacking in quantitative data by which quantitative comparison can be made, cannot be accepted as a scientific proof of the method of computation.



(*a*) SECTION *A-A* AT ELEV. 1980. (*b*) SECTION *B-B* AT CENTER

FIG. 61.—CALLES DAM.

Furthermore, as far as the writer can see, Mr. Turner's method not only assumes tensile strength where it does not exist, but takes no account of the different radii of the different parts of a dam, the varying thickness, or of any temperature effect.

Fig. 19 (*a*) does not at all resemble the actual plan of Horse Mesa Dam. If it was used in Mr. Turner's analysis and if the results are correct it would seem that the plan of an arch dam has nothing to do with the analysis. As far as the writer can see, Mr. Turner's method will show the same "relative deflections" if the dam is straight, slightly curved, greatly curved, thin, or thick; if it acts under a drop in temperature or a raise; or if it is inclined or vertical.

For these reasons he cannot accept Mr. Turner's conceptions nor his conclusions. Some rational method may be devised in the future, by which

all the involved factors in the problem can be considered and the dam analyzed as a unit, but as yet this has not been done.

Professor Constant agrees with the first three conclusions in the paper. For the fourth he presents a method of analysis which entails the use of influence lines. For circular arches of constant thickness without tension this method should be more rapid than that given in the paper.

In all the dams thus far studied by the writer it has been thought advisable to increase the abutment thickness either because of pressure in uncracked arches or to provide for the formation of adequate interior arches where tension was indicated. In the case of non-circular or irregular shaped arches, the shapes of which change with each trial computation, the influence-line method may not be so advantageous.

Mr. Hanna, as did Mr. Rich, notes the neglect of shear and develops formulas for including it. He also calls attention to the fact that the arches do not deflect radially except at the crown and that the lateral cantilevers are affected by the torsional force required to twist them to the deflected arch, and *vice versa*.

Dr. Vogt states that this effect is not appreciable. This matter was also considered by the Committee of Engineering Foundation on Arch Dam Investigation which found the effect so small that it was neglected.*

Mr. Mensch gives a clear exposition of a circular arch under normal loads and an analysis for secondary arches. There are several excellent methods and Mr. Mensch has added to them.

He states, "* * * it is a mystery to the writer why arched dams are not designed in the first place in the shape of the future secondary arches, with enlargements at the dangerous sections." One reason why this has not been done is that the erroneous belief persists that unreinforced and jointed concrete can resist tension in arched dams. Another is that it usually results in overhanging sections and the public and the profession are not "up to" that as yet. The writer believes that it will certainly be done in the future when the advantages of highly curved and flexible arches are better understood.

In excavating for the Horse Mesa abutments it was found that the arches would be much longer than assumed in the original design, which is a condition frequently encountered. The dam was then laid out again and analyzed by the trial load method by the engineers of the Salt River Valley Water Users Association. In the lower arches the new design is a radical departure from ordinary practice. These arches were materially thinned down at the points of contraflexture so that all the voussoirs, instead of just those at the crown and abutments, carried approximately the maximum stress. This thinning at areas of low stress is analogous to Mr. Mensch's suggestion for thickening in regions of high stress.

This also resulted in reducing temperature and rib-shortening stresses materially because the arches were made more flexible. Furthermore, a saving of about 12 000 cu. yd. over the original design was made. The lengthening of the arches of the original design would have required about 13 000

* *Proceedings*, Am. Soc. C. E., May, 1928, Pt. 3, p. 153.

cu. yd. additional, so that a total saving of about 25 000 cu. yd. resulted. Moreover, the arches are more evenly stressed than in the original design.

In the Calles Dam, designed by Julian Hinds, M. Am. Soc. C. E., which was completed in 1928, a part of the concrete in the tension areas at the abutments of the lower arches was omitted, as shown in Fig. 61 (*a*). The rock was simply left in place and the upper arches, while overhanging the lower ones, were actually supported on the undisturbed rock. This resulted in an appreciable saving.

Savings similar to those made at the Horse Mesa and Calles Dams can and will be made in the future by omitting the unnecessary parts of the arches. The writer believes it is practical to support the overhang, if necessary, on piers or webs. He is in complete agreement with Mr. Mensch that it would be more satisfactory if cantilever action could be eliminated. This, however, cannot logically be done with vertical dams. A section of any vertical wall will carry some water load. This cannot be reasonably neglected in considering the arch loads. Vertical elements in inclined dams can carry no water loads; the arches take it all. The problem is thus simplified.

The writer believes that inclined arch dams with tension areas eliminated and with thinner sections in regions of low stress or thicker sections in regions of high stress, will be common in the future.

Professor Cain points out the fact that the method as described neglected the friction between the arches, and that this neglected factor contributes to their strength. Dr. Vogt also goes into this matter of tangential resistance in more detail and shows that in wide dam sites it is an appreciable factor.

It is pleasing to the writer that Professor Cain so strongly emphasizes the fact that tension cannot be transmitted across cracks and ordinary construction joints. This error has been most persistently retained in other methods of analysis. The statements of Professor Cain will go far in eliminating this fallacy in the future.

The writer is indebted to Messrs. Savage and Houk for their discussion. It clearly explains the latest development of the trial load method. He believes their discussion will extend the use of this method generally.

Returning for the moment to the effect of shear, attention may be called to Fig. 50. This arch has appreciable areas in tension. Both lines of thrust (with and without shear) are outside the middle third. If the shape of the arch were modified by increasing the abutment thickness so as to permit an interior arch to form, which would be entirely in compression, the line of thrust would be within the middle third. It would be more nearly parallel to the center line. Therefore, the effect of shear would be materially less than that indicated in Fig. 50.

It is reassuring to know that in the case of the Gibson Dam the refined methods indicated no substantial changes in the results given by the original computation. Horse Mesa is the only dam designed by the trial load method, as described in the paper, which is built and under full load. The water surface at Calles is about 50 ft. from the top. Horse Mesa is one of the very highest of dams and probably the thinnest, for its height, in the world. It shows no sign of distress and has carried its full load for several years.

The Stevenson Creek Dam was analyzed by the trial load method soon after it was first proposed. In this analysis (see Fig. 62) the effect of shear was not included in either arches or cantilevers. It was assumed that the concrete was incapable of taking tension; that the modulus of elasticity was 2 000 000; and that there was a temperature drop of 20° Fahr. The load tests on the actual dam were conducted in such a way that they did not include appreciable temperature changes.

The maximum arch stress computed by this analysis was 1 600 lb. per sq. in. The maximum given in the Report of the Committee of Engineering Foundation on Arch Dam Investigation* is about 1 100 lb. A change in temperature between 8° and 3.5° gave a stress of about 100 lb. per sq. in., or a total of 1 200 as against 1 600 as given by the analysis for a temperature drop of 20 degrees. It is seen that this crude analysis, based on assumptions considerably at variance with conditions at the time of testing, gave conservative results.

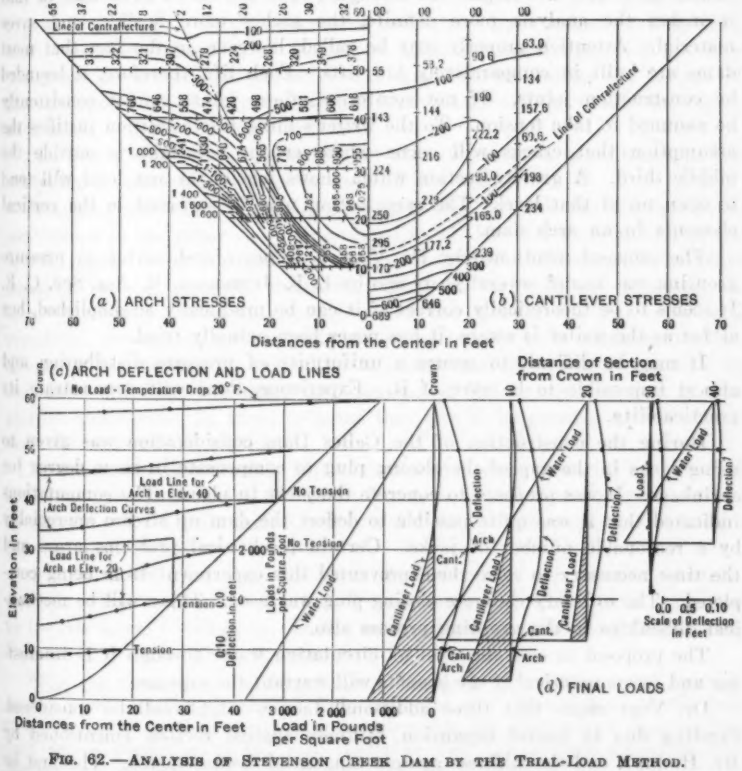


FIG. 62.—ANALYSIS OF STEVENSON CREEK DAM BY THE TRIAL-LOAD METHOD.

* *Proceedings*, Am. Soc. C. E., May, 1928, Pt. 3, p. 141.

Mr. Noetzli notes that the trial load method has been used in the design of the Horse Mesa, Gibson, and Owyhee Dams. To this list should be added the Calles Dam, previously mentioned. This is a very thin dam with a slight overhang in the central section, as shown in Fig. 61 (*b*).

Mr. Noetzli states:

"* * * it is perhaps indicated that future efforts should be directed toward alterations in the type of structure, such as would eliminate, in part at least, * * *, some of the uncertainties which are inherent in * * * the thick arch dam".

With this statement, the writer is in complete accord.

Dr. Vogt's discussion is constructive. He points out, as does Mr. Noetzli, that, once a crack has formed, the occurrence of adjacent cracks in the concrete is not probable. The writer thinks this point is well taken, and that it constitutes an uncertain element in considering "secondary arches". He also believes that removing the material down to the surface of the secondary arch does not impair the strength; it is a distinct advantage in that it makes the analysis more definite, the arches more flexible, and saves material. Attention properly may be called, however, to the fact that most dams are built in comparatively low lifts. Each lift, therefore, is bounded by construction joints. If not reinforced, these joints cannot consistently be assumed to take tension. To the writer's mind this condition justifies the assumption that cracks will occur whenever the resultant is outside the middle third. A gravity section which shows tension at any level will tend to open up at that level. The same action may be expected in the vertical elements in an arch dam.

The proposal mentioned by Dr. Vogt to improve arch action by pressure grouting was made* several years ago by L. R. Jorgensen, M. Am. Soc. C. E. It seems to be theoretically correct if it can be practically accomplished, but as far as the writer is aware, it has never been actually tried.

It may be difficult to secure a uniformity of pressure distribution and almost impossible to be sure of it. Experience alone will demonstrate its practicability.

During the construction of the Calles Dam consideration was given to using jacks in the gap of the closing plug to compensate in some degree for shrinkage. It was proposed to concrete the jacks in place. The computations indicated that it was quite possible to deflect the dam up stream appreciably by a reasonable number of jacks. Certain mechanical problems arose and the time necessary to solve them prevented this experiment from being completed. The ordinary concrete closing-plug was used. There will be mechanical difficulties in the grouting process also.

The proposal to cool the dam by circulating water through it is interesting and seems practical if the benefits will warrant the expense.

Dr. Vogt states that three additional factors might well be considered. Bending due to lateral expansion, stiffness against torsion (mentioned by Mr. Hanna), and the stiffness against tangential displacements. The first he

* *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 316.

believes will usually be negligible. In two cases he found that torsional resistance reduced the stresses slightly and he, therefore, recommends that this factor be neglected except perhaps for very thick dams. He states that resistance against tangential displacements is of considerable importance at wide dam sites, and gives mathematics for an approximate solution, together with the results of an interesting model test to confirm his conclusion.

His statement that multiple-arch dams may possibly be built without buttresses is startling. It is not evident to the writer how the usual multiple-arch section could be deprived of its buttresses, stood up vertically, and at the same time be safe from sliding even if the arch thrusts could be theoretically carried across the various units.

Regarding the cases in which gravity abutments failed and the arches stood, it should be stated that one of these dams, the Moyie, was completely reinforced by 20-lb. rails on 2-ft. centers, both horizontal and vertical.* The other, Lake Lanier, was reinforced for the top 12 ft.* of its total height of 60 ft.,‡ or 20 per cent. At the end of the arch where the abutment failed, these 12 ft. comprise about 45% of the height. The reinforcement could not help stiffening the arches against tangential displacement during the failure of the abutments. If the arches had not been reinforced the tangential resistance alone might not have saved them.

Mr. Floris expresses impatience that the writers gave some formulas for analyzing arches. He then gives some of his own.

Summing the results of the discussion it may be said that the first three conclusions in the paper are unquestioned by all but Mr. Turner. As to the fourth, nearly every one has pointed out omissions and suggested improvements in the mathematical treatment. This was expected and is welcomed. It was distinctly stated in the paper that the method was not considered complete. Several of the omitted factors were specifically mentioned. The discussion has shown how tangential resistance and shear effect may now be included. Dr. Vogt has dealt with foundation deformation so that it may also be considered. In time, it seems that it will be possible to include all pertinent factors.

In conclusion it is frankly admitted that at present the trial load method is tedious and cumbersome. However, it may be stated that a graphic method developed by Mr. Hinds indicates that a complete solution may be made in a fraction of the time now required.

It is believed that the trial load method gives a reasonably accurate analysis. The conception of vertical and horizontal elements is supported by the Stevenson Creek tests. In the report of the Committee of Engineering Foundation on Arch Dam Investigations,† it is stated,

"For the most part the directions of the principal strains were approximately vertical and horizontal. This indicates that the division of the dam into imaginary vertical and horizontal elements for purposes of analysis not only serves convenience, but also indicates that the stresses found in this manner will generally be close to the principal stresses."

* *Engineering News-Record*, Vol. 97, 1926, p. 616.

† *Proceedings*, Am. Soc. C. E., May, 1928. Pt. 3, p. 193.

An inspection of the plans of the dam designed by this method will show that they are economical in material. The discussion has shown that the method determines the maximum stresses conservatively. Further development of the method is expected to result in more accuracy and increased economy both in material and in time of designing.

For small thin dams with large central angles the application of the trial load method may not be necessary. For such dams the formulas developed by Professor Cain are thought to be adequate. For high thick dams and especially for those with small central angles and irregular profiles the writer thinks the use of the trial load method is advisable. His thanks are extended to those who discussed this paper.

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Paper No. 1713

HYDROSTATIC UPLIFT IN PERVIOUS SOILS*

By H. de B. Parsons,† M. Am. Soc. C. E.

With Discussion by Messrs. F. R. Harris, J. C. Meem, Eugene E. Halmos, T. R. Lawson, J. Vipond Davies, Lazarus White, T. Kennard Thomson, Daniel E. Moran, Alexander V. Gallogly, Allen Hazen, Charles Terzaghi, Frank S. Bailey, Charles R. Gow, E. L. Chandler, Martin J. McPike, Jacob Feld, J. Albert Holmes, E. W. Lane, and H. de B. Parsons.

Synopsis

This paper describes an original research to determine the hydrostatic uplift on the base of a structure, founded on a pervious soil, when there is no loss of head due to velocity of flow. Engineers have not been in agreement on this question, some believing that the pressure due to the hydrostatic head acts on the full area of the base, and others that the pressure acts only on a portion of the area because the remaining portion is in intimate contact with the soil.

The paper records the results of 218 tests, not including trial tests, nor those made to standardize the apparatus. The soil materials used were sand, gravel, and clay. The tests were conducted with both low and high hydrostatic heads. The size of the base area, the head, and the unit loading on the soil were varied sufficiently to demonstrate that the results are independent of these factors. Shallow depth of soil under the test base, however, does affect the results.

The effective area, or that portion of the base on which the uplift acts, is stated in percentage of the total base area. The test results show that this effective area is approximately 90 to 100%, according to the test conditions.

* Presented at the meeting of May 2, 1928.

† Prof. Emeritus, Rensselaer Polytechnic Inst.; Cons. Engr., New York, N. Y.

The apparatus used and the methods of conducting the tests are fully described and illustrated.

Introduction

There is little information on record regarding hydrostatic uplift in pervious soils. Although some tests have been published,* a search revealed a paucity of data, but a desire for more knowledge on this subject.

Divergent opinions are held by engineers on the magnitude of the hydrostatic uplift force acting on the base of a structure built on pervious soils. When the head is relatively high, the resulting uplift becomes an important consideration in the design and construction of dams, piers, caissons, and similar structures.

Obviously, there is no hydrostatic uplift under a structure founded on rock when water cannot enter the joint between the base and the rock surface. However, should water reach the base of the structure through a fissure or water-bearing seam, there would be uplift on that portion of the joint to which water could obtain access. This uplift would act only on that portion of the base area where close physical contact with solid rock did not exist.

Engineers now give recognition to hydrostatic uplift, when making calculations for important dams founded on pervious soils, and various assumptions are made to insure the safety of the structure. Ordinarily, an engineer selects one of the following assumptions according to his idea of the conditions obtaining at the site:

1.—Allowance for uplift due to total head acting on the full width of base.
2.—Allowance for uplift due to total head at the up-stream edge, which is assumed to diminish uniformly to zero at the down-stream edge.
3.—Allowance for uplift due to a portion of total head (say, three-fourths, two-thirds, or one-half) at the up-stream edge which is assumed to diminish to zero at the down-stream edge.
4.—When the down-stream edge of the base is below tail-water level, the uplift at this edge is taken at tail-water head instead of at zero, as in Assumptions (2) and (3).
5.—When conditions are favorable for a tight joint at the up-stream edge, then uplift is assumed as constant over the width of the base due to the head at the down-stream edge.
6.—Modifications of these assumptions are made to comply with the designer's idea for safety, especially when drainage ducts are provided.

The question arises whether or not it is ultra-safe to assume the uplift pressure as acting on the total area of the base, both for uniform intensity of uplift and also for varying intensity due to a flow of water through the soil beneath the base. Some engineers have favored the hypothesis that water under head enters between the particles of a pervious foundation soil, and, in consequence, the uplift pressure acts only on that portion of the base which is exposed to water. Under this hypothesis, the remaining portion of the base area, being in direct contact with the particles of the foundation soil, is not

* *Transactions*, Am. Soc. C. E., Vol. LXX (1910), p. 352.

subject to uplift because the contact, between base and soil, prevents the water from reaching this portion; and, it is claimed, the effective area would have some relation to the voids in the foundation material.

In the opinion of other engineers, uplift pressure acts on the total area of the base. Under this hypothesis, the uplift pressure acting on the base of a structure is, first, a direct hydrostatic pressure on the base area exposed to the water; and, second, an indirect pressure through the soil particles in contact with each other and with the base.

The writer undertook some experimental research work in order to obtain information regarding these hypotheses, and this paper records the results of the tests made.

Some of the smaller tests were made in the writer's office. In collaboration with Thomas R. Lawson, M. Am. Soc. C. E., apparatus was made from designs prepared by the writer, and set up in a laboratory of Rensselaer Polytechnic Institute, Troy, N. Y. Funds for the tests at Rensselaer were made available through the kindness of the Trustees from the Laflin Fund for research work. The writer also designed other apparatus, built at his own expense, which was erected in a laboratory of Stevens Institute of Technology, Hoboken, N. J. There were two series of experiments—one with low heads and one with high heads.

Low-Head Tests

Tests were made with low hydrostatic heads acting on a glass jar and on metallic cans, embedded in sand. At the instant the jar or can commences to rise from the sand, the hydrostatic uplift, acting on the bottom, just equals the weight of the jar or can, plus the sand friction against its side. It was necessary, therefore, to determine this sand friction, which was done as follows: The can to be tested was placed on a support which rested on the bottom

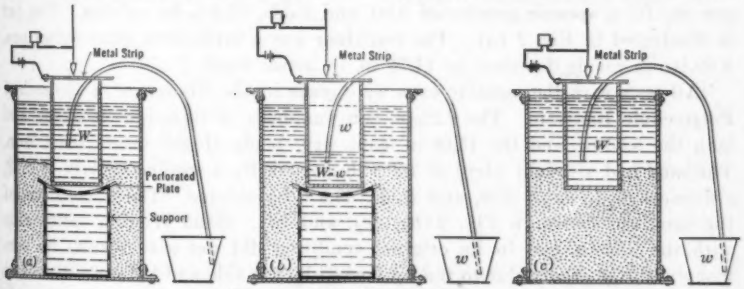

Fig. 1.—Sand Friction Tests.

of a containing vessel. This support had no top, and the can was placed on a perforated plate so that water could freely reach the bottom of the can. The supporting contact area between can and perforated plate was made small so as to be negligible (see Fig. 1 (*a*)). The support was filled with water, and wet sand was placed in the container to a predetermined height on the side of the can, and hand-packed. The can then was filled with water to give it additional weight, and afterward the container was filled with water to the

same level as the water inside the can. The outside head was carefully measured from water surface to bottom of can. To indicate the first movement of the can, a plumb-bob was suspended just clear of a metal strip across the top of the can, and wired through a battery and a buzzer. The water inside the can was slowly siphoned until the can commenced to rise (Fig. 1 (*b*)). The water siphoned from the can was weighed, to determine the weight of water left in the can at the instant of movement.

This procedure is expressed mathematically as follows:

Let U = hydrostatic uplift on the bottom of the can.
C = weight of empty can and metal strip.
C_1 = weight of water having same volume as submerged material of can.
W = original weight of water in can.
w = weight of water siphoned until can starts to float.
F = sand friction against side of can.

Then,

$$U < W + C + F$$

After siphoning w at the instant the can begins to float,

$$U = W - w + C + F$$

Also,

$$U = W + C_1$$

Equating the values of U and solving for F:

$$F = w - (C - C_1)$$

in which, $(C - C_1)$ is the weight of the can in water.

Sand Friction Tests.—All tests in the writer's office were made with a glass jar. Ordinary silica sand was used, having a weight (dry) of 112 lb. per cu. ft., a specific gravity of 2.80, and voids, 35.5% by volume. The jar is illustrated in Fig. 2 (*a*). The container was a cylindrical glass aquarium, 8.69 in. in inside diameter by 14.75 in. in inside depth.

All tests with the metallic cans were made in the laboratory of Rensselaer Polytechnic Institute. The 9.22-in. can was made of tinned sheet metal, and both the 5.53-in. and the 15.09-in. cans were made of galvanized sheet iron. The sand had a weight (dry) of 100.8 lb. per cu. ft., a specific gravity of 2.43, a fineness modulus of 2.09, and voids, 33.5% by volume. The dimensions of the cans are shown in Fig. 2 (*b*), (*c*), and (*d*). Sand friction tests made with the 9.22-in. can in its original condition did not give results in harmony with the others, due to the lip formed by the side and bottom. The tests always showed greater friction and were rejected as unreliable. The bottom recess then was filled with plaster of Paris as shown in Fig. 2 (*c*), and the recorded tests were made with the can in this condition. The container for all the cans was a steel cylinder, 36 in. in inside diameter by 32½ in. in inside depth.

Twenty-nine sand friction tests were recorded, consisting of six with the 3-in. glass jar, six with the 5.53-in. can, eight with the 9.22-in. can, and nine with the 15.09-in. can. Many other tests were not recorded because they were considered doubtful due to faulty manipulation or conditions of can that pre-

cluded accurate results. The results of the tests did not show any material difference in friction due to the materials of which the cans were made, namely, glass, tin, or galvanized iron.

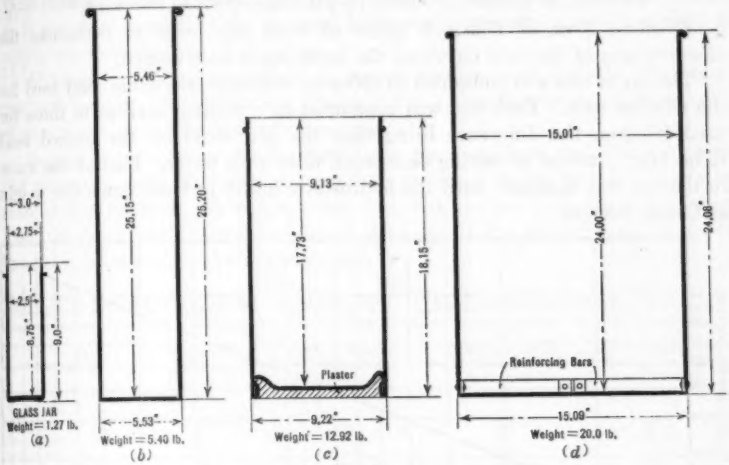

FIG. 2.

After determining the friction, the result was reduced to pounds per square inch of side surface of can embedded in sand. The results, as averaged for similar conditions, are given in Table 1.

TABLE 1.—AVERAGES OF TWENTY-NINE SAND FRICTION TESTS.

Depth that can was embedded in sand, in inches.	Friction, in pounds per square inch, of side surface of can in contact with sand.
0.5	0.01625
1.0	0.02338
2.0	0.03078
3.0	0.01616*
4.0	0.02636*
6.0	0.04700
8.0	0.04748
9.0	0.05655
12.0	0.05972
15.0	0.06790

* Doubtful.

The frictions were then platted on logarithmic paper (Fig. 3) and a mean line was drawn through the points, after omitting as doubtful the tests at 3-in. and 4-in. depths. The two discordant friction results were caused by a low one occurring at each depth, which made the averages too small. For all the uplift tests with the jar and cans, the friction was determined by reading the unit friction from the mean line (Fig. 3) and multiplying by the surface of the side in contact with the sand. This method eliminated slight irregularities due to manipulation. In order to find the mean line of friction (see Fig. 3), find the center of gravity, C, of all tests and the centers, A and B,

above and below C, respectively. The mean line passes through these three points. Tests at 3-in. and 4-in. depths of the can embedded in sand were omitted as doubtful results. The formula for the mean line is:

$$\text{Friction, in pounds} = 0.0227\ (\text{depth embedded, in inches})^{0.3977}$$

Effective Area of Base.—A series of tests was made to determine the effective area of the base on which the hydrostatic uplift acted.

The jar or can was embedded to different depths in the same sand used for the friction tests. Each test was conducted in a manner similar to those for sand friction, the difference being that the can stood on the packed sand (Fig. 1(*c*)), instead of resting on a stand filled with water. Part of the water in the can was siphoned until the hydrostatic uplift just overcame the weight and sand friction.

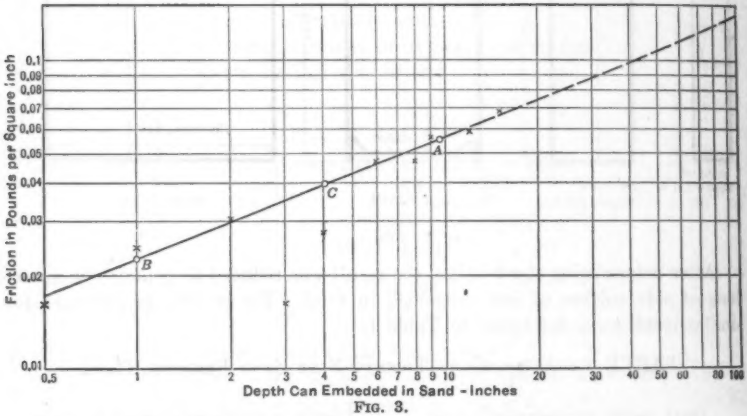

FIG. 3.

This procedure is expressed mathematically as follows:

Let A = outside area of base of can.
a = effective area of base on which hydrostatic uplift acts.
h = hydrostatic head on outside base of can.
W = weight of water in can.
w = weight of water siphoned until can just begins to float.
C = weight of empty can and metal strip.
F = sand friction against side of can.

All measurements were in inches and pounds, and the weight of 1 cu. in. of water was taken at 0.0361 lb. Then:

$$\text{Hydrostatic uplift} = 0.0361 \times h \times a$$

and,

$$\text{Resisting forces} = W - w + C + F$$

When the uplift just overcomes the resistance, these two expressions are equal. As all values are determined by the test measurements, except a, the effective area of base can be calculated. Then:

$$\frac{a}{A} \times 100 = \text{percentage of base area on which hydrostatic uplift acts.}$$

Thirty-three tests were made with wet sand, hand-packed in the container, namely, eight with the glass jar, six with the 5.53-in. can, ten with the 9.22-in. can having the bottom recess filled with plaster of Paris, and nine with the 15.09-in. can having the bottom reinforced on the inside with small angle-bars, as it was discovered that the external hydrostatic pressure caused a deflection which was measureable.

Although an effort was made to secure similar conditions in each test, the variations in the results are probably due to manipulation and difficulty of accurately recording the low pressures.

Results of tests under similar conditions were averaged, and Table 2 gives the summary, with the tests arranged in order of depth that the can was embedded in the sand. It will be noted that with the low hydrostatic pressures used, the depth of the sand bed below the base of the cans did not seem to have any influence on the effective area.

TABLE 2.—Effective Area of Base with Wet Sand Packed in Container.

Can tested.	Diameter of can, in inches.	Sand depth below base, in inches.	Depth can was embedded in sand, in inches.	Average effective area in percentage of base.
Glass jar	3.0	6.0	0.0	97.6
Glass jar	3.0	6.0	0.5	96.5
Glass jar	3.0	6.0	1.0	90.5
Glass jar	3.0	6.0	2.0	89.6
Metal can	15.09	7.5	3.0	98.2
Metal can	5.53	6.5	4.0	88.0
Metal can	9.22	11.0	4.0	99.7
Metal can	5.53	6.5	6.0	90.8
Metal can	9.22	11.0	6.0	86.7
Metal can	15.09	7.5	6.0	97.8
Metal can	5.53	6.5	8.0	81.8
Metal can	9.22	11.0	8.0	85.2
Metal can	9.22	11.0	9.0	82.5
Metal can	15.09	7.5	9.0	94.5
Metal can	15.09	7.5	12.0	92.7
Average of all tests				91.4
Average of tests omitting glass jar				90.6

A similar series of tests was made with the same sand hand-packed and tamped in a dry condition around the can. After filling the can with water, the container was filled on top of the dry sand to the same level and the test was conducted by siphoning water from the can as before. It was found that time had to be allowed for the water to saturate the sand, or the results would not be concordant.

The results are given in Table 3, being the averages of eighteen tests, namely, seven with the 5.53-in. can, six with the 9.22-in. can with plastered bottom, and five with the 15.09-in. can. The effective area in percentage is somewhat smaller than that obtained with the wet sand packing. It is possible that the results were affected by the time element mentioned.

High-Head Tests*

The apparatus consisted of a container made from a piece of extra heavy steel pipe, 12 in. in inside diameter, having a blind cast-iron flange screwed

* Made in laboratory at Stevens Institute of Technology, Hoboken, N. J.

on the bottom. A cast-iron companion flange was screwed on the top, to which was bolted a specially designed cover fitted with a stuffing-box and packing gland. A solid steel contact rod, 3.99 in. in diameter by 60 in. long, with the bottom end machined to a smooth surface at right angles to the axis, passed through the stuffing-box. The gland was packed with square-section steam piston-rod packing. There was an air-cock in the cover, and the container was tapped for a water-pipe connection and diametrically opposite for a pipe leading to a pressure gauge. The equipment is shown in Fig. 4. The contact rod weighed 216 lb., and had a base area of 12.53 sq. in. When supported by a soil, the unit load was, therefore, 17.24 lb. per sq. in., equivalent to 1.24 tons per sq. ft.

TABLE 3.—Effective Area of Base with Dry Sand Packed in Container.

Can tested.	Diameter of can, in inches.	Sand depth below base, in inches.	Depth can was embedded in sand, in inches.	Average effective area in percentage of base.
Metal can.....	5.53	6.5	4.0	90.1
Metal can.....	9.22	11.0	4.0	95.8
Metal can.....	5.53	6.5	6.0	88.5
Metal can.....	9.22	11.0	6.0	74.0
Metal can.....	15.09	7.5	6.0	93.9
Metal can.....	5.53	6.5	8.0	86.1
Metal can.....	9.22	11.0	8.0	67.2
Metal can.....	15.09	7.5	9.0	86.2
Metal can.....	15.09	7.5	12.0	88.2
Average of all tests				85.6

In these experiments, the soil to be tested was first put in the container and packed. The rod was then brought down into firm contact with the soil, and water was allowed to enter the container very slowly, while the air found exit through the cock in the stuffing-box flange. When the container was full of water and the air-cock closed, the pressure rose slowly. When the uplift was sufficient to balance the weight of the rod and to overcome the stuffing-box friction, the gauge pressure was recorded. This point was indicated by a buzzer wired to a plumb-bob and a copper strip lashed to the rod. The first movement of the rod could be detected before the buzzer sounded, by watching the light under the plumb-bob point. The device proved to be very sensitive.

Before each test, the difference in level between the base of the contact rod and the gauge center was measured. The gauge pressure, as registered, was corrected for this difference in head. The gauge also was calibrated before using, and care was taken to note that the contact rod was vertical by sighting along the plumb-bob cord.

Immediately after determining the hydrostatic uplift gauge pressure, a friction test was made. This was done by raising the contact rod just clear of the soil, so that its base was in complete contact with the water. The hydrostatic pressure required just to raise the rod was then recorded, as before.

Let A = base area of the contact rod.

a = effective area of rod, that is, the area on which uplift acts.

W = weight of rod.
F = friction of stuffing-box packing.
P = corrected hydrostatic gauge pressure when rod, resting on the soil, just commences to rise.
p = corrected hydrostatic gauge pressure when rod, being in complete contact with water, just commences to rise.

Then,

$$Pa = W + F = pA$$

or,

$$\frac{a}{A} = \frac{p}{P}$$

Then, the ratio of effective area to total base area, expressed in percentage, is $\frac{p}{P} \times 100$.

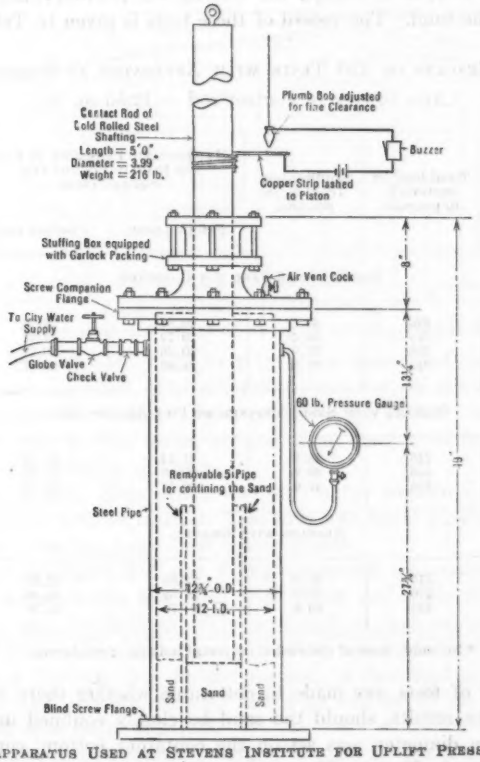

FIG. 4.—APPARATUS USED AT STEVENS INSTITUTE FOR UPLIFT PRESSURE TESTS.

Tests were made to determine not only the hydrostatic uplift, but also whether a variation in the depth of soil bed, beneath the base of the contact rod, would cause any change in the uplift pressure.

Contact Tests with Sand.—The first test was made without sand, but with the contact rod resting directly on the cast-iron bottom of the container. Although the base of the rod was machined smooth and the surface of the cast-iron bottom was rough, there was direct contact between the metals. Therefore, the effective area recorded was that portion of the area of base exposed to water. After this test, the contact rod and container head were removed. Wet sand was then put in the container to the required depth, leveled, and packed by hand. After the head was replaced, the sand was further packed by ramming with the contact rod.

The sand was brown silica building sand, having a weight (dry) of 107 lb. per cu. ft., a specific gravity of 2.5, and voids, 31.2% by volume.

The tests were made by varying the depth of the sand bed below the base of the rod, from about 1 in., then to 6 in., and then to 12 in. They were conducted to find, first, the hydrostatic uplift when the rod base was in contact with the sand bed; and, second, the stuffing-box friction when the rod was just clear of the sand. The record of these tests is given in Table 4.

TABLE 4.—RESULTS OF 110 TESTS WITH APPARATUS AT STEVENS INSTITUTE.
(Area of base of contact rod = 12.53 sq. in.)

Sand depth below base of rod, in inches.	Total load on material, in pounds.	Stuffing-box friction, in pounds.	Hydrostatic Pressure at Base of Rod, in Pounds per Square Inch.		Average effective area, in percentage of base area.
			Friction test.	Contact test.	
Results with Sand not Confined.					
0.0*	216	44.7	20.81	23.76	87.5
0.875	216	43.6	20.72	21.97	94.2
6.5	216	57.9	20.26	20.77	97.5
12.25	216	59.2	21.96	22.27	98.6
Results with Sand Confined by Pipe Around Rod.					
1.687	216	48.9	21.14	22.19	95.2
7.125	216	40.7	20.49	21.00	97.6
12.25	216	41.6	20.56	20.63	99.4
Results with Gravel.					
1.5	216	47.9	21.06	21.55	98.0
6.5	216	54.9	21.62	21.92	98.7
13.5	216	56.6	21.76	21.57	100.0

*No sand; base of rod rested on metal bottom of container.

Another set of tests was made to determine whether there would be any difference in the results, should the sand be closely confined under the rod. A pipe, 5 in. in diameter, was set on the container bottom, concentric with the rod, as shown in Fig. 4. Wet sand to the required depth was put in this pipe and packed by using the rod as a ram. The area of the base of the contact rod is 12.53 sq. in. These tests also are recorded in Table 4.

Contact Tests with Gravel.—Gravel was put in the container and tested in a similar manner. The gravel consisted of smooth, rounded, quartz pebbles, varying in size from ⅛ to ½ in., with an occasional stone ¾ in. in longest dimension. It had a weight of 97 lb. per cu. ft., a specific gravity of 2.62, and voids, 40.76% by volume. Thirty-seven tests were made. The results are recorded in Table 4.

The relation of the records is made clearer by inspection of Fig. 5, in which the results given in Table 4 are platted and curves drawn through the points. It will be noted that the hydrostatic uplift acted on practically the full area of the base when the sand depth beneath the base exceeded 17 in. Also, as the depth of the sand bed was decreased, the effective base area became less. It will be noted, also, that the confinement of the sand, by the 5-in. pipe placed around the 4-in. rod, did not materially affect the results. With gravel, the effective area of base was larger for shallow depths of bed than with sand.

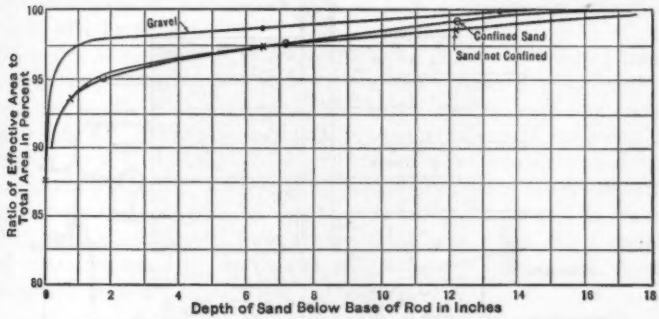

FIG. 5.—RELATIVE RESULTS OF TESTS AT STEVENS INSTITUTE.

Every test was repeated from four to seven times, and the averages of similar tests were recorded. The differences, however, in the gauge pressure readings of similar tests, were slight, as shown in Tables 6 and 7. There were 110 tests made in this series, not including many trial tests to see that the apparatus was working properly.

Contact Tests with Clay.—Clay was put in the container, and an effort made to test it in a similar manner. The clay was fairly pure in quality and of the kind used for top dressing of tennis courts.

In a damp condition, the clay firmly supported the contact rod, but it flowed when the container was filled with water and while the contact rod was used as a ram for packing. The rod sank at each ram, until its base touched the bottom of the container. An effort was made to confine the clay closely by using the 5-in. pipe around the rod, as described previously. The result was the same, namely, the clay flowed up and out of the 5-in. pipe and acted like sand with quick behavior.

When tested with hydrostatic pressure, the rod always sank into the clay bed (whether confined or not) below its original starting point as soon as the pressure was released. The results (such as they were) indicated that the effective area was nearly a 100% of the base area.

Tests Made at Rensselaer Polytechnic Institute

The apparatus consisted of a riveted steel cylinder, 36 in. in diameter by 32½ in. deep, as shown in Fig. 6. The bottom was stiffened by 2-in. angle-bars and the sides were reinforced by steel bands 3¼ in. by ⅜ in. The cover, fitted with a stuffing-box and gland, was bolted to the cylinder, and could be removed for placing sand in the container. The contact rod was a piece of 12-in. extra strong pipe, turned to 12½ in. in outside diameter and filled with concrete. A 3-in. pipe extended above the 12-in. pipe for centering the concrete weights. Three concrete weights were made, each approximately 3 by 3 ft. by 9 in. thick, with projecting steel reinforcement bars to facilitate handling. The container was tapped for a water pipe, to which was fitted a pressure gauge.

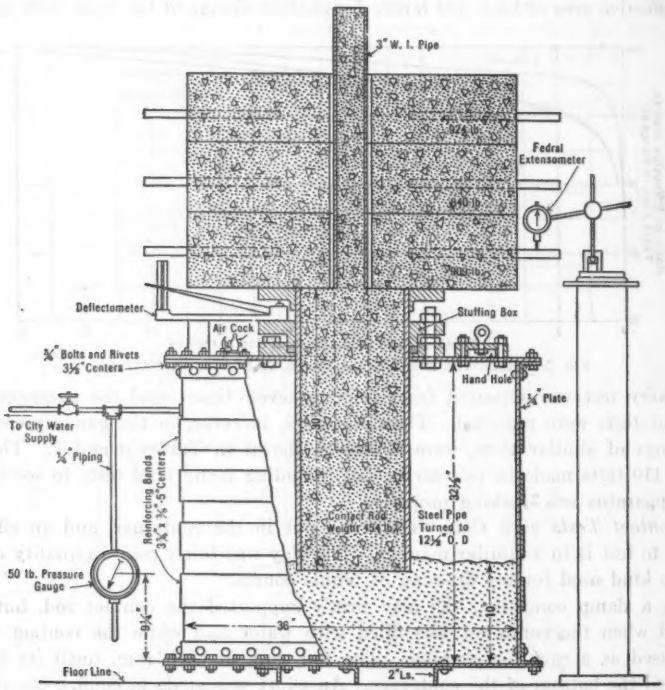

FIG. 6.—APPARATUS USED IN UPLIFT PRESSURE TESTS, AT RENSSELAER POLYTECHNIC INSTITUTE.

The contact rod weighed 454 lb. and had a base area of 122.7 sq. in. The concrete weights were 909, 840, and 924 lb., respectively. When resting on the sand for testing, the unit loadings were, therefore, 533 lb. per sq. ft. without the weights; 1 600 lb. with the lower weight; 2 585 lb. with the two lower weights; and 3 669 lb. with all three weights.

The tests were conducted in a manner similar to those made at Stevens Institute, and the mathematical formula was the same. The pressure gauge was calibrated, and the readings as corrected were recorded. An extensometer was used to indicate the first movement of the contact rod, and a deflectometer to record any change in position of the cover of the container due to the heavy internal hydrostatic pressure.

Tests were made to determine, not only the hydrostatic uplift, but also whether a variation in soil loading would cause any change in the uplift pressure.

Contact Tests with Sand.—Wet sand was put in the container to a depth of about 10 in., leveled and hand-packed. The contact rod, when loaded with weights, settled into the sand; but prior to each test, the depth of sand directly under the contact rod base was measured.

The sand was smooth or water-worn silica sand, having a weight (dry) of 116.7 lb. per cu. ft., a specific gravity of 2.83, a fineness modulus of 2.50, and voids, 34% by volume.

The record of the tests is given in Table 5. It should be noted that the loading in these tests was heavy, under the three weights, being more than 1.8 tons per sq. ft. of bearing area. While the greatest care was exercised to secure accuracy, the results may seem to have unwarranted variation in the stuffing-box friction. This variation in friction was probably caused by the centers of gravity of the heavy concrete weights not being always directly in the axis of the contact rod. Any slight eccentricity would tend to make the rod bind in the stuffing-box. As the recorded results are the averages of several tests (confirmed by trial tests), the percentage of base area found to be effective is no doubt close to the true figure.

TABLE 5.—Results of Twenty-Eight Tests with Apparatus at Rensselaer Polytechnic Institute.

(Area of base of contact rod = 122.7 sq. in. Sand packed wet.)

Sand depth below base of rod, in inches.	Total load on material, in pounds.	Stuffing-box friction, in pounds.	Hydrostatic Pressure at Base of Rod, in Pounds per Square Inch.		Average effective area, in percentage of base area.
			Friction test.	Contact test.	
9.25	454	262.6	5.84	5.86	99.7
10.00	1 363	353.6	13.99	14.51	96.5
9.63	2 203	319.7	20.56	20.75	99.2
9.25	3 127	619.0	30.53	30.39	100.0

The data given in Tables 1 to 5 are averages of the results found by repeating similar tests. The results of a few tests are given in more detail in Table 6, with low heads, and Table 7 with high heads.

The observations recorded in these tables show the closeness of repeated tests, which were not always made on the same day. The closeness of similar tests also shows the fairness of using averages to eliminate personal variations in manipulation and observation. Furthermore, the personnel of the

test parties was changed, and the results obtained by the various parties were in harmony.

TABLE 6.—LOW-HEAD TESTS.

$(0.0361 \times h_0 \times a = W - w + C + F)$

Can tested.	Depth embedded, d.	Friction curve value $\times A_0 \times d$, F.	Weight of can + contact plate, C.	Outside water head, h.	Weight of water in can, W.	Weight of water siphoned, w.	Percentage of base area effective.	Average percentage of base area effective.
WET SAND CONTACT.								
Glass jar, area = 7.07 sq. in.	1	0.214	1.27	8.06	1.6	1.22	91.0	90.5
	1	0.214	1.27	8.06	1.6	1.24	89.9	
Can, 5.53 in. in diameter, area = 24.02 sq. in.	4	2.74	8.05	21.50	18.15	12.25	89.5	88.0
	4	2.74	8.05	21.50	18.15	12.85	86.5	
Can, 9.22 in. in diameter, area = 66.77 sq. in.	6	8.0	14.82	16.50	37.95	28.30	81.6	86.7
	6	8.0	14.82	16.60	38.20	25.05	90.0	
	6	8.0	14.82	16.40	37.70	25.45	88.6	
Can, 15.09 in. in diameter, area = 178.84 sq. in.	9	23.10	22.65	20.40	129.7	55.03	91.6	94.5
	9	23.10	22.65	20.20	128.2	49.80	95.5	
	9	23.10	22.65	21.00	133.3	49.00	96.4	
DRY SAND CONTACT								
Can. 5.53 in. in diameter, area = 24.02 sq. in.	4	2.74	8.05	21.75	18.37	11.40	94.0	90.1
	4	2.74	8.05	21.50	18.16	12.30	89.5	
	4	2.74	8.05	21.50	18.16	12.80	86.6	
Can, 9.22 in in diameter, area = 66.77 sq. in.	6	8.0	15.57	16.20	37.22	32.50	72.5	74.0
	6	8.0	15.57	16.20	37.22	31.35	75.2	
Can. 15.09 in. in diameter, area = 178.84 sq. in.	9	23.10	22.65	20.25	128.80	65.50	83.5	86.2
	9	23.10	22.65	20.65	131.20	58.15	89.0	

SUMMARY

The preliminary testing disclosed that the "time element" entered the problem when dry sand was used. It required several hours for the sand to become saturated, and for the full hydrostatic pressure to be evenly transmitted. When using the large container at Rensselaer, as long as 18 to 24 hours was allowed for saturation. An experiment was made by having one end of a rubber tube embedded in the sand under the test base, and noting the time required for the water in the tube to reach the same level as that in the container.

To avoid inaccuracy caused by the time required for saturation, the sand used in the tests at both Stevens and Rensselaer was packed in a saturated condition.

By not embedding the base of the contact rod, the problem of sand friction was eliminated in the high-head tests. The experiments showed that:

1.—In pervious soils the area of base on which uplift acts is always a large percentage of the total base area. Although the results are generally between 90 and 100% of the total base area (with a few below the lower limit), it would not be safe to estimate an uplift in pervious soil of less than full pressure head acting on the total area of base, when there is no flow of water through the soil; nor less than the full pressure head indicated by the hydraulic gradient when there is flow through the soil.

The hydrostatic pressure acts, not only on that portion of the base area to which the water has access, but also on each particle of the soil material. Therefore, a part of the uplift pressure is transmitted to the base through the points of contact of the particles with one another and of the particles with the base.

TABLE 7.—HIGH-HEAD TESTS.

$$\left(\text{Percentage of base area effective} = \frac{p}{P} \times 100\right)$$

Total load.	Material of bed.	Area of base.	Corrected Gauge Pressure for Friction = p.		Corrected Gauge Pressure for Contact = P.		Average percentage of base area effective.
			Reading.	Average.	Reading.	Average.	
216	No material.....	12.53	20.81 20.81	20.81	23.87 23.86 23.56	23.76	87.5
216	Sand not confined..	12.53	20.26 20.26	20.26	20.77 20.77 20.77 20.77 20.77 20.77	20.77	97.5
216	Confined sand.....	12.53	20.74 20.24 20.49	20.49	21.25 21.25 20.75 20.75 21.00	21.00	97.6
216	Gravel.....	12.53	21.26 21.26 21.51 21.01 22.26 22.01 21.76 21.51 22.01	21.62	21.27 21.27 21.52 22.52 22.52 21.77 22.27 22.02 22.27 21.77	21.92	98.7
1363	Sand not confined..	122.7	13.99 13.99 13.99	13.99	14.51 14.51 14.26 14.76	14.51	96.5

In a bed of pervious material the hydrostatic pressure acts on the portions of the surfaces of the particles which are exposed to the water. A base prevents hydrostatic pressure from acting on the top surfaces of the particles in contact with it. Consequently, the resultant of the forces on all the particles beneath the base is upward. The sum of this resultant pressure acting through the particles plus the hydrostatic pressure due to the water acting directly on the base constitutes the total uplift on the base. With sufficient depth of soil, the effective area of the base is found to approximate 100 per cent. Therefore, the portion of the total uplift acting through the particles

directly on the base is equal to the hydrostatic head at the plane of the base times that part of the base area in contact with the soil. At shallow soil depths the aggregate of the uplift forces acting through the particles fails to equal the hydrostatic head at the plane of the base times the area of the base in intimate contact with the soil.

2.—The uplift is reduced when the depth of the pervious soil is shallow. The tests at Stevens showed this reduction commencing at about 16 or 17 in. of soil. These tests also showed that the uplift is less with finer soil particles at corresponding depths of soil within the limit named. The experiments made at Stevens (Table 4 and Fig. 5) were the only ones in the series designed to record the effect of variable depth of soil beneath the base. The results of tests at Rensselaer (Table 5), in which the area of base of the rod was about ten times greater than that at Stevens, showed that the average effective area in percentage of base was approximately 98.8 when the sand depth below the base was a little more than 9 in. At a corresponding depth for the tests at Stevens (Fig. 5), the effective area was about 98.0% for sand not confined, 98.3% for confined sand, and 99.2% for gravel.

3.—The variation in sizes of base areas, from 7.07 sq. in. for the 3-in. jar to 178.84 sq. in. for the 15.09-in. can, or twenty-five times, does not alter the result.

4.—The variation in hydrostatic uplift pressure per unit of area, from 0.29 lb. per sq. in. to 30.39 lb. per sq. in., or one hundred times, does not alter the result.

The profession would be benefited if engineers would report data on magnitude of uplift pressures observed in construction work.

Acknowledgments

The writer expresses his thanks to Thomas R. Lawson, M. Am. Soc. C. E., Philip D. Joynt, Jun. Am. Soc. C. E., and Messrs. C. E. Rose and E. R. Wiseman for their help in conducting the tests at Rensselaer; to Mr. George L. Quigley for his help with all the tests and especially for his aid in revising the manuscript; to Eugene E. Halmos, M. Am. Soc. C. E., for suggestions made during the work; and to Mr. Warren R. Bentley, his Office Assistant.

The writer also acknowledges his obligation to the Trustees of both Rensselaer Polytechnic Institute and Stevens Institute of Technology for space in the testing laboratories.

DISCUSSION

F. R. HARRIS,* M. AM. SOC. C. E.—The speaker notes that in his "Introduction", Mr. Parsons states that there is little information on record regarding hydrostatic uplift in pervious soils and was impressed with this as far back as 1909. However, in connection with the design and construction of Dry Dock No. 4 at the Brooklyn Navy Yard, there was considerable discussion on this subject.

It is apparent that a dry dock structure is one in which the hydrostatic uplift is of primary importance. In fact, it is the outstanding condition which influences the entire design of the work. In all dry dock structures built previous to 1909, with which the speaker is familiar, the full hydrostatic pressure incident to extreme high tide, or even in excess of this, if the apparent ground-water elevation was higher than extreme high tide, was calculated. There was considerable discussion at that period as to whether or not such uplifts were present and active. One of the theories in vogue then, which Mr. Parsons mentions, is that this pressure was lessened by a reduction in the area of the sub-grade surface in contact with water and subject to water pressure; that is, the surfaces of the subsoil in contact were thought to consist of fine particles of materials pressing against the under side of the structure which served to reduce the area subject to water contact and hydrostatic pressure. It was considered that this, in turn, reduced the total hydrostatic uplift and that this condition obtained so long as the weight of the structure was in excess of such uplift.

The speaker had some doubts as to the correctness of this theory, in fact, he did not agree with it and, therefore, the dry dock was designed to take full hydrostatic uplift. For future information, devices for recording the actual uplift were placed in the dock floor at several places. These consisted of cylinders with a lead diaphragm, the under side of the diaphragm being at sub-grade. The cylinder projected up into the concrete floor and was embedded in it. A plunger or piston with sufficient clearance so as not to bind in the cylinder, was held against the upper side of the diaphragm by a shaft with a capstan screw. On completion of the dock structure observations were made on the uplift, the piston being released for this purpose. In every case practically full hydrostatic pressure was recorded on the diaphragm. In connection with these observations there was provided, through the dock floor, pipes fitted with pressure gauges and the observed water pressure on the gauges was checked against ground-water and tidal observations, the gauges indicating full hydrostatic pressure, fluctuating with tidal changes.

Observations on the diaphragm followed the pressure-gauge observations on the pipe connections so closely that the speaker was convinced that the hydrostatic uplift was 100 per cent.

The speaker regrets that it has not, as yet, been possible to arrange these data in a suitable form for publication. The results were, nevertheless, very convincing, in indicating undoubted full hydrostatic uplift.

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J. C. MEEM,* M. AM. SOC. C. E.—The author is to be commended highly for the painstaking care with which he has endeavored to solve one of the difficult problems of applied engineering. The speaker has made somewhat similar experiments, although not, perhaps, so exhaustively, with similar apparatus, and has also co-operated with the author, at least in the early stages of this experimental work. He believes, however, that the results of the author's experiments are not conclusive as to the reduction of area of uplift pressure through soil.

The principal reason for this belief is that in order to determine the reduction of area definitely, it is necessary to establish a specific area of contact, and this contact does not appear to have been established in any of the author's experiments, due to the small area of the can or piston, compared with the relative size of the grains of sand or gravel, and to the shallow depth—especially in the case of the hydraulic piston—at which it was subjected to pressure. For instance, under a microscope this piston would have appeared as if resting on a few pinnacles rather than on a flat area. Furthermore, if the author was unable to get more than 15% contact (see Table 4) between the bottom of the piston and the bottom of the container, it is reasonable to suppose that he would not be able to establish a larger area of contact on loose, irregular particles. In the four experiments which the speaker made with hydraulic apparatus similar to that used by the author, he was able to establish a much higher percentage of contact by plunging the piston into the sand, pressing it well down, and leaving it for varying intervals until normal conditions resulted.

In order to give a better understanding of this experiment, a section of the apparatus is shown in Fig. 7(*b*). It consists of a hydraulic chamber with the piston actuated hydraulically by a pump connected by copper pipe to the chamber and carrying a gauge reading to pounds. It was found that it took about 4 lb. of pressure to raise the piston when plunged into water alone.† A table was then placed in the chamber so that sand could be packed around the piston, leaving the bottom free, and it was found that the additional pressure required to raise the piston did not register on the gauge; and frictional resistance due to this sand was assumed to be negligible or less than 25 lb. per sq. ft.

The table was then removed, the chamber was partly filled with sand, and the piston pressed into it, where it was allowed to remain for periods varying from a few minutes to 2 hours. The pump was started and the pressure required to lift the piston varied from 17½ to 25 lb. (averaging about 22 lb.), which gradually dropped to 4 lb., when the piston came out of the sand.

Even if the friction on the piston due to surrounding sand is determined by using the coefficient found by the author, it is seen that the pressure required to lift the piston from sand with which it has made contact is three or four times that required to lift it in clear water.

It should be noted that if there is 100% contact of water or pressure on the base of the piston there should also be 100% of water pressure around

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† *Transactions*, Am. Soc. C. E., Vol. LXX (1910), p. 367.

the piston; in which case, friction should be negligible under the author's theory. When a piston rests in the sand, or is only slightly depressed into it, the reaction of the pressure from the piston to the sand voids under it tends to depress the sand further and the action is wedge-like around and under its edges, gradually depressing the sand away from the piston until it has established an area sufficient to lift it. Proper contact may be established in many ways, but those most nearly approximating conditions of practice are: Plunging the piston to a sufficient depth and pressing it firmly into the sand; placing a layer of cement mortar between the piston bottom and the sand; or placing a layer of stiff clay, so that in the latter two cases the action of the water will be through the voids of the sand or gravel against the mortar or clay, raising the latter and, consequently, raising the piston. Careful distinction should be made between structures resting on soil and those resting partly on piles or caissons. In the latter cases the soil may tend to settle away from the structure and leave the area not supported exposed to the full pressure over the remaining area. This accounts for the fact that gauges

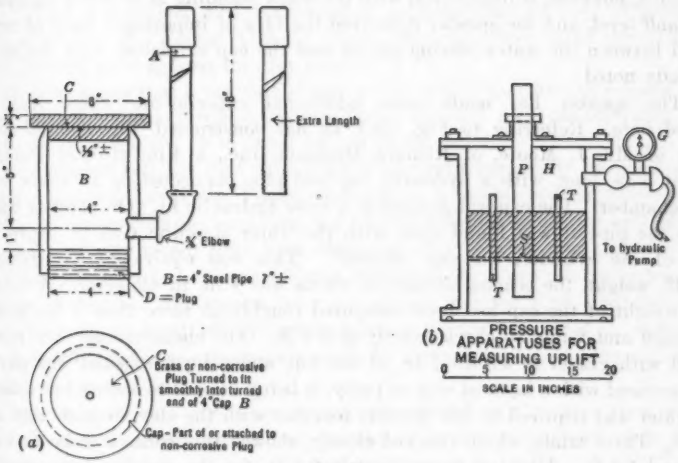

FIG. 7.

through submerged and buried structures may register full pressure, even if there may not be full area pressure, on the bottom, top, or sides of the structure. Water under pressure must eventually open up a lead if openings or outlets are caused to exist, even if the adjoining areas may be excluded from pressure by contacts extending beyond the sphere of influence. Where the structure rests on soil, however, any settlement of the soil results in a settlement of the structure, so that the ratio of uplift area is not changed.

Nevertheless, whether the structure rests on caissons, piles, rock, gravel, or soft ground, it must either be floating or supported. Where the supporting areas are of sand, gravel, or other soil, there must be areas of contact between the bottom of the structure and the soil, extending beyond the influence of

uplift, and, while the minimum sections of these areas of contact may be in various planes, the cross-section through the minimum areas represents in sum those excluded from uplift or buoyant pressure. This cannot be other than a definite fact. The author states that even if there may be areas of soil in contact with the piston or bottom of the structure, other areas of soil are pressed against these, tending to give full pressure over the whole area. This is not in accordance with actual conditions; that is, a structure, together with the soil beneath it, is either floating or it is supported; both conditions cannot be present at the same time.

It does not appear to the speaker that the question in this case, as to whether or not the pressure through soil is static or dynamic, is as important as the question whether or not the water pressure exerts itself against or through compact masses of soil.

It is obvious that if an absolute contact is made between the soil and the hydraulic cap (in the experiment referred to), the problem of relative uplift can be solved in the laboratory as well as in the field.

This, however, is impractical with the water standing at or above the gravel or sand level, and the speaker conceived the idea of imposing a layer of moist sand between the water-bearing gravel and the cap or piston, with the result already noted.

The speaker has made some additional experiments which may be noted here. Referring to Fig. 7(*a*), he has constructed, through the courtesy of Mr. J. Moore, of Richard Dudgeon, Inc., a 4-in. circular chamber about 5 in. long, with a hydraulic cap and pipe connected by an elbow with the chamber. The cap was ground to a loose hydraulic fit. On pouring water into the pipe it was found that, with the water standing 0.55 ft. above the top of the cylinder, the cap "floated". This was equivalent to lifting a 2.8 lb. weight, the bearing surface of which was 4 in. in diameter. Although the weight of the cap had been computed roughly at more than 3 lb., it was weighed and found to check exactly at 2.8 lb. The chamber was then partly filled with gravel to within $\frac{3}{4}$ in. of the top, and a layer of sand was put in and covered with a layer of clay or putty, it being desired to show what column of water was required to lift the cap, together with the clay, or both clay and sand. Three trials, which checked closely, showed this column to be between 1.1 and 1.2 ft. Allowing approximately 0.1 ft. for the weight of the sand, a column of 1.2 ft. would be exactly double the clear water column, showing 50% reduction of area due to the gravel.

In some of the later experiments with the apparatus shown in Fig. 7(*a*), there was 4 in. of sand below the cap. It would require a water column of 6 in. approximately to lift this, and about 6 in. more to lift the cap, that is, 1 ft. in all. It is not believed that friction need be considered in so small a layer of soil, with the inside faces greased, and it is thus seen that, if the water stood at 8 in. above the top of the chamber, plus the 4 in. below, this 1-ft. head should lift the sand and cap, or that a 6-in. head above the chamber should lift the cap alone, from clear water.

As a minimum of 1 ft. 8 in. was required to do this, it would seem to indicate a reduction of the pressure area of 70% on the cap alone, or 50% on the

cap and sand; that is, if the cap alone had lifted from the sand, it would appear that the pressure acted through 30% of the voids of the sand, or through 50% of the gravel voids in lifting the sand cap—which is almost certainly what happened.

Since these experiments were made, the speaker's assistant, J. C. Fisher, Jun. Am. Soc. C. E., has made several additional experiments. His findings are as follows: When the water-bearing gravel was placed within 1 in. of the top of the chamber and ½ in. of moist sand was placed over it, covered to the top by a clay cap, with the inside of the chamber greased to prevent friction and leakage, this clay cap was lifted (apparently) by a column of water 0.165 ft. above its bottom plane. The column required to lift from clear water was approximately ¾ in., so that 1⅛ in. would show 60% reduction of area pressure, or 40% effective area pressure. This experiment was repeatedly made with approximately similar results. If, as may have been the case, the sand lifted from the gravel, then the column of water, acting ½ in. lower, would have been approximately 0.20 ft., or 2.4 in., of which 1½ in. was required to lift sand and clay. This would show 40% reduction of pressure area, or 50% of pressure area effective. Mr. Fisher thinks, however, that the water acted through the thin layer of sand against the clay cap.

He also found that a can forced into a thin bed of mortar and pressed into sand to insure contact, would not rise (after setting) until its buoyancy was approximately doubled by drawing off the inside water.

The speaker believes that this latter experiment can easily be made on a large scale, and that if Mr. Parsons had gotten a mortar contact between the bottom of his can and the sand, and had allowed time for setting up and adjustment, his results would have been different. He believes also that the best way to make the uplift experiment on a large scale is to use a large piston in a hydraulic chamber and plunge the piston deeply into the sand, giving it time for settlement and adjustment before putting on pressure; that is, repeating the hydraulic chamber experiments made by the speaker* with a much larger piston and apparatus.

In each case, care was taken to get contact and, at the same time, to see that the cap or piston was as free to turn as in the case of the clear water trial.

It is probable, however, that full contact cannot be established in small areas such as this; but this experiment is believed to prove definitely that the uplift area is greatly diminished by the presence of sand, gravel, or soil of any kind.

The speaker is now practically convinced (*a*) that the area of uplift pressure on a submerged structure, buried or partly buried in soil, is reduced below the percentage of voids rather than equalling it, or tending toward 100% of the whole area; (*b*) that the pressure of water is against a mass of soil rather than through it, unless leads, orifices, or artificial voids occur; and then (*c*), unless such leads or artificial voids are made to exist, that the flow of water through soil is rather by seepage than by pressure head and, although this

* *Transactions*, Am. Soc. C. E., Vol. LXX (1910), p. 364.

head influences the flow, the influence is incidental rather than direct, and in proportion to the void characteristics of the soil.

Daugherty states:*

"Thus, in the case of a ship, for example, sunk in the mud at the bottom of a body of water, the pull, T, necessary to raise the ship is not only the weight of the ship, but also the entire volume of water on top of it."

This is probably true of all structures buried or partly buried in submerged soils which do not contain a large percentage of voids natural or artificial.

The speaker believes that pressure is rapidly dissipated when it has no outlet, just as it has been found impossible to force grout or pea gravel under pressure into natural voids; but when it has an outlet, grout soon opens up a lead and can be forced into artificial voids long distances away from the pump.

Here it may be interpolated that rust, too, will fill artificial voids rapidly, but when these are filled it will cease to function, except as a protective coating for the remaining steel or iron.

Returning to the matter of pressure through voids, the speaker, through his assistants, continued the experiments with the apparatus shown in Fig. 7(*a*) and found that when gravel was placed in the bottom of the chamber and sand to a depth of 4 in. was pressed down compactly above it, with a contact cap of grease between the sand and the piston cap, the sand was forced out of the chamber under a column equal in head to from 1.8 to 2.1 ft. This finding showed two things definitely: It took from 75 to 100% more head of water acting through gravel to raise the sand column from a gravel contact than was theoretically required to lift its own weight, together with that of the cap. That is, the weight of the cap and sand together was 2.6 + 2.8 = 5.4 lb., which, distributed over 12.56 in. of area, equalled 0.43 lb. per sq. in., or approximately a column of water of 1 ft., which would indicate a contact of from 40 to 50% between the sand and gravel. It further showed that the line of least resistance was not through the voids to lift the lighter cap which would raise in clear water at 6½ in. of column, but rather against the heavy body of sand requiring a column of from 1.8 to more than 2 ft. This, and a previous experiment,† proves definitely that where there are no artificial voids, uplift pressure is greatly diminished or absent on partly buried submerged structures.

The history and record of such structures are full of such evidence. Among typical examples may be cited: (*a*) A structure concreted to solid rock in which there are no fissures and, consequently, no uplift; (*b*) dredges flooded and sunk which do not rise when pumped out and have to be "hydraulically mined" to raise them; (*c*) hulks half buried in sand which cannot be raised through buoyancy alone; (*d*) tunnels and structures which do not rise after the soil is stabilized around them, although difficulties are sometimes experienced in holding them down during the exigencies of construction; and numberless others. This is definite evidence that theory and

* "Hydraulics," p. 33.

† A photograph of the sand column lifted in a similar experiment made by the speaker is shown in *Transactions*, Am. Soc. C. E., Vol. LXX (1910), p. 364, Fig. 22.

practice must here be in accord. Sometimes the instances cited are noted as due to suction, but there can, of course, be no suction unless the pressure is almost or entirely excluded from the otherwise buoyant area.

It is generally known that the Holland Tunnel rose more than 12 in. due to the hydraulic envelope caused during construction by the agitation into a soupy condition of the surrounding soil, and that it had to be partly back-filled to prevent flotation; and yet it cannot be doubted that when this hydraulic envelope had been replaced by grout and the surrounding soil stabilized, there could no longer be any possibility of the tunnel rising.

Light back-fill, or friction grip, even if it may be more considerable than found by the author, will not hold down a structure which has a water-jacket, or is under what may be called full buoyant pressure. The speaker agrees with the author that it is always safest to design, where practicable, against full pressure area, particularly in structures subject to heavy vibration, shallow submergence, or shallow cover. This is equivalent, however, to saying that steel structure designs should have a factor of safety; but it does not militate against the theory that there cannot be pressure and contact in the same area. The speaker wishes to thank F. L. Cranford and C. H. Locher, Members, Am. Soc. C. E., and their staff for assistance in making the experiments noted.

Eugene E. Halmos,* M. Am. Soc. C. E.—The speaker had the privilege to be in constant touch with the work done by the author in devising and making the tests and in the interpretation of the results. He believes that the method of approaching the problem and the manner of conducting the tests will be found to be above criticism and that the author's conclusions will meet with the approval of the profession.

What Mr. Parsons did not state but of which the speaker knows, is the large amount of labor and the considerable time the author has devoted to these tests and the substantial sums of money which he has expended in order to find the answer to a question the solution of which he considered of commensurate importance for the benefit of the Engineering Profession. Moreover, there was the agony of suspense lasting for days and often for weeks at a time when, due obviously to errors in technique somewhere, results contrary to reasonable expectation were recorded. These errors could not be easily detected on account of the complexity of the elements which, by the nature of the problem itself, go to make up the results of the tests. The whole apparatus and every step of the experiment had to be re-examined with painstaking care many times before all sources of error were eliminated.

Most of the experiments were conducted at Stevens Institute and at Rensselaer Polytechnic Institute. It was found that the engineering colleges, perhaps with very few exceptions, are not properly equipped to make tests of a variety of practical problems in their hydraulic laboratories. In the case under discussion, the efforts and the money of a private individual had to be utilized in the solution of a problem which is of interest to all engineers. The speaker believes that this is another instance that indicates the

* Chf. Engr., Parklap Constr. Corporation, New York, N. Y.

necessity of installing modern hydraulic laboratories in American institutions of learning.

As to the paper itself, the speaker wishes to comment on one point only which has not been emphasized by the author, probably because it is somewhat of a by-product.

In making the tests to determine the friction on the sides of the cylinders, the author has found a formula which shows that the average friction per unit of surface is proportional to the 0.4 power of the depth to which the cylinders were embedded. The total friction per linear unit of perimeter, therefore, would be proportional to the 1.4 power of the depth. It is an accepted principle in mechanics that within the limits of the pressures here considered, the friction is proportional to the pressure exerted against the surface. Therefore, the pressure of the sand around the cylinder has been shown to be proportional to about the 1.4 power of the depth, contrary to the formulas used in the design of retaining walls, dock walls, etc., which assume that the pressure of both dry and saturated earth-fill is proportional to the square of the height of the backing. In other words, the author's formula gives a much smaller pressure than that of Rankine. It seems that the difference, which is very substantial, is due to cohesional forces acting between the particles of sand used for the tests. These forces are neglected in the formulas derived to determine the pressure on retaining walls. On the other hand, when sinking caissons in saturated ground to substantial depths, practical constructors assume uniform friction along the outer surface of the caisson, the value of which they find to be independent of the depth, in other words, proportional to the zero power of the depth. The author's formula, therefore, gives values greatly in excess of the friction experienced in such cases.

In view of the foregoing, the speaker believes that it is well to emphasize the fact that the formula given by the author, while correct within the limits of his experiments and for the material utilized in his tests, should not be used for the purpose of designing retaining walls, caissons, or similar structures.

T. R. Lawson,* M. Am. Soc. C. E.—Emphasis should be placed on the care taken by the author to eliminate all possible variables and to secure conditions which will approximate those of actual practice. The 12-in. plunger used in the apparatus at Rensselaer Polytechnic Institute was a hollow cylinder filled with concrete, thereby giving a contact between a concrete base and the sand, thus simulating the actual conditions under the base of a concrete dam.

In order to prevent any trouble from settlement, no readings were taken until after the full load had been applied through the concrete weights, and a sufficient interval of time had been allowed between loading and readings until all possibility of settlement had ceased. This time interval extended from 24 to 48 hours.

When these experiments were first begun, some difficulty was experienced from the expansion of the large cylinder from internal pressure. The difficulty

* Head, Dept. of Civ. Eng., Rensselaer Polytechnic Inst., Troy, N. Y.

was made apparent by observing that, under pressure, the piston settled into the sand instead of rising. The difficulty was overcome by heavily reinforcing the outside container with steel bands. No trouble was experienced after this up to pressures of 30 lb. per sq. in.

The series of experiments were repeated a number of times under different loads on the piston, and in each instance the upward motion of the piston began at a load approximately equal to the friction of the sand on the side of the piston plus the product of the area of the piston and the pressure per square inch indicated by the pressure gauge.

This apparatus is still available for further tests, and should it appear desirable as a result of the discussion of this paper, other experiments will be attempted.

If the upward pressure on a pervious soil is 100%, as indicated by the experiments, considerable attention must be paid to cut-off walls not only on the heel to take care of the seepage under the base, but also at the toe where the full static pressure from the tail-bay will act on the base.

The results from these experiments bring forcibly to mind another very important research on which there is but little reliable data; that is, the thrust due to a saturated fill on the back of a retaining wall. While retaining walls are designed with a fairly accurate knowledge as to the thrust due to a dry fill, very little information is available to the engineer as to the effect of saturating this fill with water. Information of this character would be particularly useful in the design of harbor and dock walls where the effect of tide on the back of the wall is an important factor.

J. Vipond Davies,* M. Am. Soc. C. E.—The speaker notes with interest that engineering investigators differ on the matter of hydrostatic uplift in soil. When he first came in touch with the question of the construction of tunnels under the Hudson River, he was told that there were two schools of engineering thought on the subject. One was that any tunnel built under the Hudson must of necessity float up to the surface by uplift, and then float down the river. Members of the other school insisted that it was absolutely certain that any tunnel built under the river would sink in the comparatively soft silt and that would be the end of it.

So far as the speaker knows, every tunnel built under the Hudson River has been weighted, at some time or other during its construction period, to keep it from lifting. The Hudson and Manhattan Tunnels were all weighted by the excavated silt. This was left on the floor of the tunnel until such time as the superincumbent load or flow of the soil, comprising the river silt, squeezed in on the tunnel lining and gave weight enough to resist the buoyancy of the tunnel itself. Subsequently, the tendency was toward a downward settlement notwithstanding that the tunnels are much less in weight than the displaced soil.

It is a fact that any dredging in the bed of the river causes uplift in the tunnel below. There is no doubt about that; it is a perfectly measurable quantity. During the recent dredging of the 2 000-ft. channel in the Hudson

* Cons. Engr.; Pres., Jacobs & Davies, Inc., New York, N. Y.

River to a 40-ft. depth plane, there was a very definite uplift in all the tunnels under the river, caused by the removal of that superincumbent load.

Many engineers are familiar with the fact that all tunnels in Hudson River silt pulsate with the daily rise and fall of the tide, sinking as the tide rises and the pressure on the silt bed increases, and lifting as the tide falls and the pressure on the bed of the river reduces.

It is equally certain that any tunnel under the Hudson can be lifted bodily by pumping stiff puddled clay, or some material like clay, through the invert. The tunnels normally remain stable and stationary under the river; but there is no doubt that a tunnel can be lifted if that were desired for any reason. It can be raised and uplifted by high-pressure pumping of a solid material.

As to the distribution of the pressures over areas, the speaker has never had the assurance to do otherwise than consider 100% of the hydrostatic uplift pressure uniformly distributed over the entire area of structures that were likely to be in contact with water.

As an example, in the Hoboken Terminal of the Hudson Tunnel System, a station structure was designed as a box with complete arched inverts, partly in silt and partly in sand. In that case the entire structure was designed to give 100% loading against hydrostatic uplift. There were parts in which the weight of the structure itself would have been less than the aggregate load necessary; and in those sections the speaker had the space under the platforms filled with concrete to increase the loading against uplift. These localized areas were (*a*) at the east end of the station where the silt was very soft, being near the water-front; and (*b*) where the concourse floor above the main structure reduced the amount of back-fill forming part of the superincumbent load.

Any man would be quite brave who attempted to design structures of this character, without allowing fully for the entire hydrostatic uplift, uniformly distributed over the whole area.

Lazarus White,* M. Am. Soc. C. E.—As a result of experiments performed in 1910,† the speaker was rather inclined toward the belief that there was not 100% hydrostatic uplift on structures built in pervious soils. Mr. Parsons' tests were made with the expectation of rather confirming the same views and when the results were found to be inconsistent with his ideas they were studied very carefully. The speaker has been unable to find flaws in these later experiments.

The question arises, How can one reconcile the older tests with those made by the author? They were both honestly made and they are radically different in their conclusions. The speaker believes that the latter were conducted under truly static conditions and the former under dynamic conditions. In the older experiments there was a flow of water in the sand and a great loss of pressure.

As Spencer has said "there is some truth left in every error". However, because there is an uplift on 100% of the area it does not follow that there

* Pres., Spencer, White & Prentis, Inc., New York, N. Y.

† *Transactions*, Am. Soc. C. E., Vol. LXX (December, 1910), p. 352.

is 100% pressure, except in the case of a truly static condition, which does not usually exist in construction work. It is, therefore, necessary to multiply a 100% area by a pressure which is less than the full hydrostatic pressure.

Suppose, for instance, that there is 100% pressure on a concrete floor in a coffer-dam. Through leakage there is a great reduction of head, so that the total uplift is much less than the maximum which can be computed. For that reason those who have constructed coffer-dams incline to the belief that uplift does not act on the full area. The true explanation is that it is acting on the full area, but not with an intensity corresponding to 100% of the difference of head between the inside and outside water levels.

It is well known that 20 ft. of head can be supported by 1 ft. of concrete, provided the foundation material is well under-drained and the leakage is pumped away. Of the concrete, 1 ft. will suffice because allowing that flow gives a tremendous loss of head between inside and outside. In quite a number of cases the need of allowing for 100% uplift has been obviated by permanent under-drains and pumps. This is a natural condition as much as anything else of which the engineer may take advantage if he sees economy in it.

T. KENNARD THOMSON,* M. AM. SOC. C. E.—It seems to the speaker that when subsoil materials are taken into a laboratory they are no longer subsoil materials. They can never again be put into a condition duplicating that in which Nature had placed them or giving similar test results. For instance, in each and every case there is a difference in the arrangement of the more or less natural cementing material, and there is no way of telling how many thousands of pounds of pressure Nature may have exerted during the placing, or shortly thereafter.

In Western New York State the speaker has seen hardpan, or glacial drift, that in some former age had been placed under an unknown depth of water, at an elevation of more than 1 200 ft. above sea level. One might almost as well pulverize concrete and then expect it to act in a laboratory as if it still were concrete.

An accident in one of the Hudson River tunnels would be appalling. However, the fact remains that all these tunnels are lighter than water and are subject to slight daily variations in level, rising and falling inversely as the tide; although, of course, the theoretical hydrostatic uplift should depend entirely on the difference in elevation between the top and bottom of the tunnel and not on the total depth of the water.

What probably happens is that the effect of the tide re-acts more quickly on the roof than on the bottom of the tunnel, allowing the water to be squeezed away from the base, so that when the tide rises the tunnel is forced down into the silt, reversing the process as the tide falls. While no one can prove that the tunnels will ever be wrecked, no one can prove that they will not. It would be worse than foolish to make this statement—except as a warning against constructing future Hudson River tunnels on the same basis as the present tubes. Beyond a peradventure of a doubt, perfectly safe tunnels can be built, and for much less money, in the Hudson River.

* Cons. Engr., New York, N. Y.

A circular caisson, carried to hardpan in Lower New York, with concrete 4 ft. above the cutting-edge in the working chamber, was lifted by the water pressure and had to be removed, at a considerable loss. On another occasion a large rectangular caisson resting on rock—at a depth of 19 ft. of water in the Susquehanna River, with the 6-ft. working chamber filled with concrete and concrete also placed above the deck—was lifted by an unusually high tide and had to be towed away and destroyed. In that case if there had been 1 ft. more of concrete on the deck, the mass would have been heavier than the water displaced. Each and every foundation is apt to have certain conditions not found elsewhere and all the laboratory tests in the world would not eliminate the necessity of using one's best judgment. It would appear, therefore, that the only safe rule for uplift is to use 100% of the hydrostatic head.

DANIEL E. MORAN,* M. AM. SOC. C. E.—The speaker would like to emphasize two points. When water is moving through sand there are static conditions and dynamometric conditions, and they must not be confused. Moreover, there is capillary action. If the base of a layer of fine sand is wet, the water will rise in the sand by capillary action. In laboratory experiments on sand there are radically different conditions than those found in a natural deposit.

The second point is that in making various tests in the laboratory it became very evident to the speaker that if he loaded sand in a cylinder, the forces transmitted from the loaded piston to the platen went partly through the sand and partly through the cylinder. He then substituted for the platen two pistons, one on the top and one on the bottom, with only a relatively small volume of sand in the cylinder.

When the speaker applied the load on the top piston and tried to measure it on the bottom piston, part of the load was being transmitted through the sand and part through the walls of the cylinder containing the sand. Therefore, he had to be very careful in reaching any conclusions.

The author's experiments were undoubtedly accurate and designed to secure accurate results. The only mystery to the speaker is that he did not get 100 per cent.

ALEXANDER V. GALLOGLY,† ASSOC. M. AM. SOC. C. E.—Some years ago the speaker was engaged in the construction of an earthen dam. The reservoir was formed by cutting off the top of a hill and depositing the excavated material around the edge to form an enclosure about ¼ mile wide and ½ mile long. The excavated material was ideal, consisting of about two-thirds sand and one-third clay and boulders.

The dam was built in 8-in. layers and rolled with a 10-ton grooved roller. No stones larger than 8 in. in thickness were included. The resulting material had nearly the nature of concrete. This part of the dam (which formed the inner portion) was called the "impervious embankment", and it proved to be quite impervious. It was about 180 ft. thick at the base, 30 ft. at the

* (Moran, Maurice & Proctor), New York, N. Y.
† Care, Carpenter & Duffy, Mamaroneck, N. Y.

top, and it had a maximum height of 42 ft. Excavated material, including boulders, was placed around the outside in 2-ft. layers without rolling.

About four years after the reservoir had been filled with water, the work of cutting through the dam for the purpose of tapping the lake was undertaken. The material was found to be dry and acted like concrete, there being no curve of leakage, such as that described in textbooks. This condition continued until the original ground surface was reached and passed. When the bottom of the trench had been excavated to a point about 1 ft. below the original surface, a small pool of water about 2 ft. in diameter appeared. This was bailed out, and the material was excavated with a shovel until the water began coming through the fine sand lenses in a manner similar to that observed in the excavations outside the reservoir. The level at which this water appeared was 21.5 ft. below the surface of the water in the lake. As the excavation proceeded, this condition gradually developed over the entire area of the bottom and sides of the trench below the level of the original surface under the dam.

As the trench was being excavated and just before the water appeared, a peculiar hissing sound was heard. This was found to be caused by air which was being forced out ahead of the water. If upward pressure was present to any extent, why did it not push up the soil in the bottom of the trench? Did not the presence of the trapped air cause an additional weight to act against the hydrostatic head? During the construction of the gallery, the seepage into the trench amounted to only 300 gal. in 24 hours. This was removed by bailing.

The speaker believes that the conditions surrounding a foundation placed in water-bearing material, can be compared to a U-tube closed at one end, in which the water in the open leg is balanced by the foundation itself, together with that portion of the material under the foundation in which air is trapped. It is known that all water contains small air-bubbles which tend to collect under a flat surface.

In the experiments described by the author, the apparatus seems to illustrate the conditions surrounding a river caisson only. In such a case the upward pressure would equal the full hydrostatic head plus the upward pressure caused by the weight of the semi-liquid material penetrated by the caisson. The experiments do not seem to cover all the cases of foundations constructed in pervious materials below ground-water level.

ALLEN HAZEN,* M. AM. SOC. C. E. (by letter).—This paper presents a careful experimental investigation of one of the fundamental conditions of dam construction. It is no less valuable because it affords a complete confirmation of ideas long held by many members of the Society. It should lead to the discard of all less complete allowances.

The effect of uplift from water pressure is not confined to material below the base of masonry dams. It may also exist in any part of the structure where the masonry is pervious unless adequate interior drainage is provided. In cold climates frost may cause a temporary sealing of outlets at the lower

* Cons. Engr. (Hazen, Everett & Pirnie), New York, N. Y.

face of the masonry and this, in turn, may result in development of full pressure in the full thickness of a dam.

The effect of uplift is to reduce the stability and strength of the dam. It tends first, to overturn it, and, second, to take off weight and reduce the friction and resistance to sliding.

The resistance of natural materials to sliding is not known with any satisfactory degree of definiteness. There is reason to think that with some softer rocks, such resistance is not in proportion to height and weight; and for high structures it may be much less than has sometimes been assumed. The Panama Canal slides furnish an illustration, from an entirely different field, of the low coefficient of friction of soft rocks at high pressures. Experience with harder materials and lower unit stresses may not be a safe guide for soft rock. Masonry dams have failed by sliding more frequently than by overturning.

Looking at the matter broadly, the problem of dam design needs to be managed with a strong hand. Dams and reservoirs must play an important part in the future development of this country. They must be made strong enough and safe enough so that the people who live near them as well as those who build them and benefit by them will be assured of their safety.

In the past, some dams have been built of ample proportions strong enough for all contingencies. There are many dams in the United States that would stand if filled with a liquid twice as heavy as water. These dams are safe. Even earthquakes do not seriously shake or touch them. On the other hand, there are many dams that would not stand such a test. Too many dams have failed to reach the point of actual stability.

Engineers speak of a factor of safety, but the term needs to be defined more closely. In a steel structure the factor of safety may mean the ratio between the stresses at the assumed loading and the ultimate strength of the steel, but no one thinks that a load corresponding to the ultimate stress could be safely applied. Engineers speak of a factor of safety of 4, but really they believe that something like double the assumed load would probably be held.

It may be somewhat the same with dams. When a designer states that a factor of safety of 2 was used—based no doubt on some rather arbitrary assumptions—it may mean that the real factor of safety is much less than 2. It may turn out that it is little, if any, more than 1. It certainly does not mean that a liquid twice as heavy as water could be safely held.

It may be earnestly hoped that some one will set up an authoritative policy of dam design that will lead to the construction of dams that would be stable and safe if subjected to pressure from a liquid much heavier than water. If this could be done, it would tend to restore confidence in dam design and construction; and even if some of the works to be presently constructed, should cost somewhat more than they otherwise would, some such policy is needed and will tend to the ultimate benefit of all concerned.

CHARLES TERZAGHI,* M. AM. SOC. C. E. (by letter).—The hydrostatic uplift acting through sands on the bottoms of seals, cellar floors, and other

* Prof., Foundation Eng., Mass. Inst. Tech., Cambridge, Mass.

structures, is so simple and clear an hydraulic phenomenon that it is far more difficult to account for the attitude of engineers toward the uplift than it is to compute its intensity.

The theoretical aspects of hydrostatic uplift acting through sands were convincingly presented in 1912 by A. Franzius,* and it is almost inconceivable that the opponents of Mr. Franzius† could fail to realize the weight of his arguments. Gaede, in 1917,‡ and Engesser, in 1919,§ computed the intensity of the uplift acting through sand, assuming that the area of contact between the individual sand grains is determined by the Hertz formula (area of contact between curved surfaces pressed against each other). The computation led to the conclusion that the hydrostatic uplift acting on a surface in contact with the sand should range between 97% and almost 100% of the hydrostatic uplift exerted by water, provided the modulus of elasticity of the sand grains ranges between 100 000 and 500 000 kg. per sq. cm., and that the pressure acting on the sand does not exceed 10 tons per sq. ft. The fraction of the area over which the hydrostatic uplift does not act should, according to theory, be independent of the size of the grains. These theoretical results agreed with the results of the experiments performed in 1886 by Brennecke,‖ in 1887, by Forchheimer,¶ in 1916, by Schaper,** and in 1917, by Busemann.†† They also are in very good agreement with the results of the tests performed in 1926 by Mr. Tsin Hsiao, at the Massachusetts Institute of Technology, under the supervision of the writer, and in moderately satisfactory agreement with those now published by the author.

All the tests, including those of Mr. Parsons, were based on the same principle, namely, measuring the hydrostatic uplift at the instant when the forces acting on the submerged body balanced those acting downward. Because of this fact, Busemann claimed in 1917 that the test results thus obtained do not represent the hydrostatic uplift which acts on the base of a pier before an actual uplifting occurs. However, his arguments can be considered conclusively invalidated by the results of Gaede and the computations of Engesser.

As a matter of fact, the data and assumptions on which the theory is based, are so plain and the range of possible error associated with theory is so small (if compared with the importance of the errors which are apt to be connected with laboratory tests in that particular field), that in all cases where the test results are in conspicuous contradiction with theory, investigators should first direct their attention toward finding errors in the testing method. It should be kept in mind that a real contradiction between test results and theory—that is, a contradiction which cannot be accounted for by errors of observation—would mean that there is an error in the fundamental principles of hydrostatics, and there are not many sciences which rest on so stable a foundation.

* *Zentralblatt der Bauverwaltung*, 1912, p. 583; 1913, p. 617.
† *Loc. cit.*, 1912, pp. 522 and 617; and 1913.
‡ *Loc. cit.*, 1917, p. 501.
§ *Loc. cit.*, 1919, p. 443.
‖ *Zeitschrift für Bauwesen*, 1886, p. 101.
¶ *Zentralblatt der Bauverwaltung*, 1887, p. 314.
** *Loc. cit.*, 1916, p. 514.
†† *Loc. cit.*, 1917, p. 205.

There is such a contradiction, for instance, between the results of the Engesser theory and the conclusions reached by Mr. Parsons concerning the effect of the depth of the permeable stratum on the intensity of the hydrostatic uplift. According to theory, the thickness of the layer of sand between the base of this structure and the impermeable bottom below the sand should be without any effect on the hydrostatic uplift, provided it is at least equal to several times the diameter of a sand grain. According to Mr. Parsons, the uplift is reduced when the depth of the pervious soil is shallow. The writer believes that the apparent influence of the thickness of the layer was due to the air content of the sand beneath the base of the can. Undoubtedly, Mr. Parsons has done his best to exclude this source of error, but every one who has had experience with sand experiments knows that it is almost impossible to exclude the air entirely. Thus, for instance, when saturating a sand by allowing the water to enter through a permeable base, thus giving the air a chance to escape freely in an upward direction, it was found by weighing and by computation that, within the saturated space, more than 30% of the voids remained filled with air and the degree of saturation increased very slowly as time passed. If it has no possibility of escaping in an upward direction because of the presence of the bottom of the can, it is hardly to be expected that the air will ever disappear completely. One of the possible effects of the air content on the hydrostatic uplift, if measured by high-head tests, can be explained by Fig. 8. The dotted line indicates the size of an air bubble at the outset of the test. As soon as the water pressure goes up, the bubble becomes smaller and this, in turn, requires the water to flow toward the bubble. Then the head required to overcome the resistance against flow causes an apparent increase in the pressure required to lift the can, because the true resistance consists of both the weight of the can and the head required to direct the flow toward the shrinking bubbles.

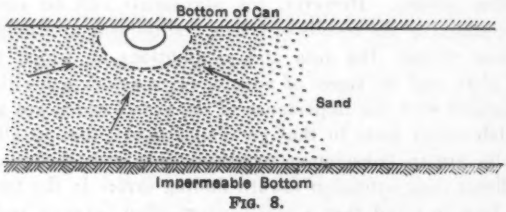

FIG. 8.

In the tests which were performed in 1926 by Mr. Tsin Hsiao, at the Massachusetts Institute of Technology, the diameter of the can was 5 in. In order to reduce the effect of any trace of air on the test results, the sand was placed on top of a layer of coarse sand (grain size, from 2 mm. to 6 mm.), so that the distance between the bottom of the can and the top of the coarse sand was nearly equal to 1 in. The results of the tests were as follows:

Grain size, in millimeters.....	2.00	1.41	0.84	0.35	0.10
Effective area, in percentage....	99.25	99.85	99.50	98.50	99.25

Considering these figures and the fact that the results obtained by Mr. Parsons are as far from being as consistent with each other as those obtained by

Brennecke in 1886 and by Forchheimer in 1887, the writer hesitates to accept his conclusions concerning the effect of the thickness of the layer on the intensity of the hydrostatic uplift. The writer is inclined to consider this effect exclusively as a result of various imperfections of the testing method. However, as a whole, Mr. Parsons' tests furnish figures that are at least of the same order of magnitude as those which ought to be obtained according to the laws of hydrostatics. Hence, it is hoped that they will serve their purpose of demonstrating to the skeptic members of the Engineering Profession (for the sixth time since 1886) that the uplift acting through sand really is very nearly 100% of the uplift acting through water, provided the underground consists of almost cohesionless sand. If Mr. Parsons succeeds in achieving this purpose, his efforts will certainly have been well worth while.

Far more difficult is the question concerning the hydrostatic uplift acting through fine silt and clay. For measuring the uplift in such materials, no satisfactory experimental method has been found thus far—Mr. Parsons' method included. The theory of Engesser also loses its validity because every soil grain is surrounded with a layer of adsorbed water which has, for all intents and purposes, the properties of a solid. No hydrostatic uplift can be transmitted through this layer, the thickness of which is certainly not more than $\frac{1}{20\,000}$ mm.; it may be thinner. For sand grains, the presence of this layer can be neglected. Hence, for such materials, Engesser's theory is accurate enough. However, for grains the size of very fine silt and clay particles, these layers are apt to become an item of increasing importance.

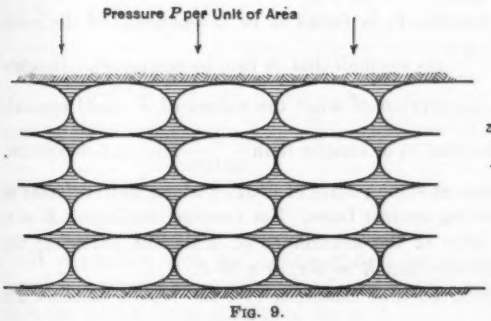

FIG. 9.

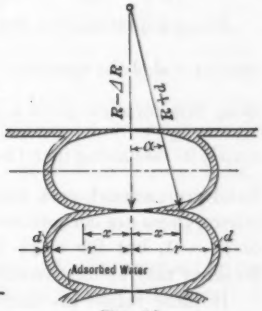

FIG. 10.

For evaluating the influence of the layers of adsorbed water on the hydrostatic uplift, assume that the bottom of a can is supported by lens-shaped grains arranged as shown in Fig. 9. The grains touch each other at their flat sides. Each grain is surrounded with a skin of adsorbed water with a thickness, d (Fig. 10). Since, at the bottom of the can, the area exposed to the uplift is smaller than that between the grains, the computation must be made for a section beneath the first row of grains (Fig. 10).

Let E = the modulus of elasticity of the grains;
r = the radius of their vertical projection;

n = the shape factor;
$R = n\,r$ = the radius of curvature of their surfaces of contact;
P = the pressure per point of contact;
p = the downward pressure exerted by the can on the sand (including hydrostatic uplift);
x = the radius of the area over which no hydrostatic uplift can act;
F = the proportion of a unit of area of the base upon which no hydrostatic uplift can act; and,
ΔR = the shortening of the radius of curvature, R, due to the pressure, p.

The number of points of contact per unit of area of the base is $\frac{1}{4r^2}$. Since the angle, α, is very small, the radius, x, is almost equal to,

$$2\,R\,(d + \Delta R) = 2\,n\,r\,(d + \Delta R)$$

For R, the formulas of Hertz give the term:

$$\Delta R = 0.77\,\sqrt[3]{\frac{P^2}{E^2\,R}} = 1.94\,r\,\sqrt[3]{\frac{p^2}{n\,E^2}}$$

The area over which the hydrostatic uplift cannot act is,

$$F = \frac{x^2}{4\,r^2} = \frac{1.57\,n}{r}\left(d + 1.94\,r\,\sqrt[3]{\frac{p^2}{n\,E^2}}\right) = 1.57\left(\frac{d\,n}{r} + 3.05\,\sqrt[3]{\frac{n^2\,p^2}{E^2}}\right)$$
$$= 1.57\,(C_1 + C_2) \qquad (1)$$

in which, $C_1 = \frac{d\,n}{r}$ and $C_2 = 3.05\,\sqrt[3]{\frac{n^2\,p^2}{E^2}}$.

From Equation (1) the value, F, is found to be independent of the grain size, provided the quantity, $\frac{d\,n}{r}$, is so small that it can be neglected. In addition, this formula gives a conception of what the values of F could approximately be, assuming that d is equal to or smaller than $\frac{1}{20\,000}$ mm. = 0.000005 cm. Sand grains touch each other at some points of contact along flat surfaces; at others, there are corners resting against faces. For average conditions, $R = r$, or $n = 1$, but for clays (due to the abundance of scale-like particles) the designer may assume tentatively that $R = 5\,r$, or $n = 5$.

If these values are introduced into Equation (1), the results obtained are as shown in Table 8.

The figures assembled in Table 8 (A) show that there is no possibility of accounting for an effect of the presence of sand or gravel on the hydrostatic uplift in excess of about 1%, provided the laws of hydrostatics are valid. The differences between 0.99 and Mr. Parsons' figures can safely be considered a measure for the efficiency of his testing arrangement.

In connection with the figures quoted in Table 8 (B) it should be remembered that the films of adsorbed water which exclude the transmission of hydrostatic uplift, also produce an adhesive bond between the particles. In other words, the greater the pressure-reducing effect of the films, the greater should

be the cohesion of the material. This conclusion agrees with the empirical fact that a microscopic sand ($n = 1$, with small values of F) has very little cohesion if compared with a clay ($n = 5$, with large values of F). Hence, the greater its cohesion, the greater should be the pressure-relieving effect of the presence of the substratum on the hydrostatic uplift. Absence of cohesion is a safe indication for almost 100% uplift.

TABLE 8.

		Bulky particles, $n = 1$.	Scale-like particles, $n = 5$.
(*A*) Grain Size, 0.2 Mm. ($r = 0.01$ Cm.)			
$p = 0$	C_1	0.000500	0.00250
	C_2	0	0
	F	0.000784	0.003930
Effective area, in percentage	....	99.9	99.6
$p = 5$ kg. per sq. cm.	C_1	0.000500	0.00250
	C_2	0.001424	0.00416
	F	0.003024	0.0104
Effective area, in percentage	....	99.7	99.0
(*B*) Grain Size, 0.002 Mm (Limit Between Silt and Clay, $r = 0.0001$ Cm.)			
$p = 0$	C_1	0.05000	0.2500
	C_2	0	0
	F	0.0784	0.3930
Effective area, in percentage	....	92.2	60.7
$p = 5$ kg. per sq. cm.	C_1	0.05000	0.25000
	C_2	0.00142	0.00416
	F	0.081	0.398
Effective area, in percentage	....	91.9	60.2

In connection with the sealing of foundation pits, prior to unwatering the sealed space, there is still another factor which requires serious consideration. It is the relative permeability of the seal.

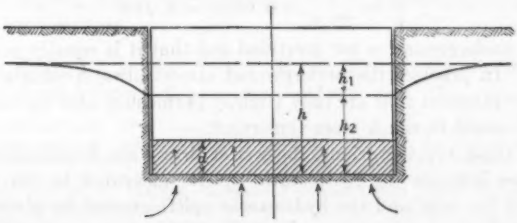

Fig. 11.

Fig. 11 represents a cross-section through a round or a square foundation pit surrounded by a water-tight enclosure and sealed by a somewhat permeable bottom slab with a thickness, d.

Let F = the area of the bottom of the pit;
k_1 = the coefficient of permeability of the seal;
k_2 = the coefficient of permeability of the underground;
h = the hydrostatic head which would act on a perfectly impermeable seal; and,
h_1 = the loss of head due to percolation through the seal.

According to the law of Darcy the quantity of water, Q, which percolates through the seal is equal to,

$$Q = F\, k_1 \frac{h_2}{d}$$

On the other hand, the loss of head, h_1, due to the quantity, Q, flowing through the underground toward the seal is (according to Forchheimer) approximately equal to,

$$h_1 = \frac{Q}{4\, k_2 \sqrt{\frac{F}{\pi}}}$$

Since $h_1 + h_2 = h$,

$$F\, k_1 \frac{h - h_1}{d} = 4\, k_2\, h_1 \sqrt{\frac{F}{\pi}}$$

The relative loss of head, $\frac{h_1}{h}$, that is, the apparent decrease of the hydrostatic uplift due to percolation, is therefore, equal to,

$$\frac{h_1}{h} = \frac{1}{2.26\, d \frac{k_2}{k_1} \frac{1}{\sqrt{F}} + 1}$$

Suppose that the seal is ten times less permeable than the underground; that is, $k_1 = 0.1\, k_2$; that F_1, the area of the pit, is 100 sq. m. (about 1 100 sq. ft.); and $d = 1$ m. (3.3 ft.). Under these conditions, the drop in head due to percolation would be,

$$\frac{h_1}{h} = \frac{1}{3.26} = 0.307 = 30.7\%$$

For $k_1 = 0.01\, k_2$,

$$\frac{h_1}{h} = \frac{1}{23.6} = 0.0425 = 4.25\%$$

provided the underground is not stratified and that it is equally permeable to a great depth. In practice the underground almost always contains layers and seams of silty material that are only slightly permeable, and the corresponding drop of head would be much more important.

From all these considerations, the writer draws the following conclusions: The difference between the hydrostatic uplifts measured by Mr. Parsons on the bottom of the cans and the hydrostatic uplift exerted by plain water may essentially be due to various imperfections of the testing method. The same

seems to be true for the apparent effect of the depth of the pervious soil on the intensity of the hydrostatic uplift. The apparent contradiction between theory, according to which practically 100% of the hydrostatic uplift should act through cohesionless materials, and engineering practice which shows that the uplift through such materials is sometimes very much smaller, may essentially be due to the pressure-relieving effect of seepage through the seal, seepage through joints in the enclosure, and the adhesion between the seal and the enclosure of the pit. On the other hand, if the underground has an appreciable cohesion, the low value of the uplift may be due, wholly or partly, to the physical character of the water enclosed in very narrow spaces.

Knowing by experience the great amount of time and painstaking labor required to perform an investigation of the type described by Mr. Parsons, the writer wishes to express his appreciation, and it is hoped that the paper will serve its purpose of clearing up the prevailing misconceptions concerning hydrostatic uplift.

FRANK S. BAILEY,* ASSOC. M. AM. SOC. C. E. (by letter).—The author's researches have provided a welcome addition to the fund of engineering knowledge. The experiments were repeated so many times and the results interpreted so carefully that there seems to be no question as to the reliability of the conclusions. They should bring to an end the uncertainties and differences of opinion which may have hitherto existed.

Now that Mr. Parsons' experiments have been published, it is to be expected that records of similar experiments or of measurements of uplift pressures on various structures will be added. By coincidence some records of this nature have appeared,† describing uplift measurements made by the U. S. Bureau of Reclamation on several dams. Two of these records—the Colorado River Dam, on the Grand Valley Project in Colorado, and the Percha Dam, on the Rio Grande Project in New Mexico—show upward pressures under dams which are founded on gravel.

Records of uplift pressures on the Pinhook Dam, at Maquoketa, Iowa, and on the Island Park Dam at Dayton, Ohio, show that both dams rest on sand and gravel foundations.‡ These experiments showed that, unless the bed of the stream or reservoir above the dam was sealed by silt, the uplift pressure on the bottom of the dams on sand and gravel foundations varied from nearly full head at the up-stream end to the tail-water head at the lower end, diminishing more or less closely according to the "line-of-creep" theory. Considering the results of the author's experiments and the corroborative evidence recently published, it appears that his conclusion as to full hydrostatic head in pervious soil is entirely justified.

CHARLES R. GOW,§ M. AM. SOC. C. E. (by letter).—While it may be true that the author's conclusions are not entirely new, they are so definitely

* Structural Designer, New England Power Constr. Co., Boston, Mass.

† See paper entitled "Upward Pressures Under Dams; Experiments by the U. S. Bureau of Reclamation," by Julian Hinds, M. Am. Soc. C. E., in this volume of *Transactions*.

‡ See the discussion by E. L. Chandler, M. Am. Soc. C. E., on the paper by Julian Hinds, M. Am. Soc. C. E., entitled "Upward Pressure Under Dams; Experiments by the U. S. Bureau of Reclamation," in this volume of *Transactions*.

§ Pres., The Gow Co., Inc., Boston, Mass.

confirmatory of previous experience along similar lines that there is no longer any valid reason for the assumptions frequently made that ground-water pressures exert substantially less than 100% of the theoretical pressure due to their heads.

A question in this connection which may be entitled to consideration is that of the reduced pressure applied by a structure on the soil due to this hydrostatic uplift. A building with a water-tight basement situated below water level would apparently suffer a reduction in the amount of weight normally applied to the soil by the quantity of water displacement represented and, in many instances, this may be a considerable factor justifying somewhat higher load values than would ordinarily be used.

Likewise, it is to be noted that account is frequently taken of the weight of soil removed in the process of excavating a deep basement in determining the net increase in unit loading which the structure imposes on the supporting stratum. The usual practice has been to calculate the weight of material removed according to its weight in free air. If, however, the excavated material in its natural state is in whole or in part below the ground-water level, it may be assumed that the actual weight removed in the process of excavation is less than this amount due to its immersion in water. Of course, if hydrostatic uplift is assumed to reduce the weight of the structure this factor need not be considered since the two effects offset one another.

While the title of the paper refers especially to pervious soils, the author discusses briefly some of his experiments with clay instead of sand. As a matter of fact, it is doubtful if there are in Nature any actually impervious soils. Experiments which have been made seem to indicate that even the densest clays will permit some passage of water, although at an extremely low velocity rate. Under these circumstances there would seem to be no good reason for differentiating between the so-called pervious and impervious soils with respect to the ultimate effect of hydrostatic uplift. There are, however, certain aspects of the problem as it relates to the so-called impervious soils which are entitled to mention.

When a basement is constructed within an excavation made in a clay soil, there is danger that surface water may be admitted into the porous back-fill around the outside of the basement walls which may upon occasions supply temporarily a pressure at the level of the under side of the basement floor-slab equal to that of a water-head of the full depth of excavation. Usually, it is assumed that a more or less water-tight connection is made between the floor masonry and the supposedly impervious soil upon which it rests and to a certain extent this is probably true. One can never be definitely sure, however, that water pressure from such a source as that referred to will not find its way into the seam between the masonry and earth and thereby exert its full uplifting influence.

An instance of this sort occurred a few years ago in connection with a mercantile building in Boston, Mass. The basement floor was laid directly on a soil of the type generally recognized as impervious in character. The space surrounding the basement wall was back-filled as is more or less customary with material of porous character consisting of waste plaster, bricks,

and the like, with the result that when a heavy rain occurred shortly after the completion of the building, the voids in this loose back-fill were completely filled with water up to the surface of the ground. A few days later the basement floor which had not been designed against uplift was forced upward over the greater part of its area as a consequence of the admission of water operating under this head.

On another occasion the clear-water basin of a large filtration plant in the vicinity of Boston was founded on a stiff blue clay which showed no observable evidence of water percolation through it. After the completion of the structure a heavy downfall of rain occurred leaving a number of deep pools of water in some of the unfilled trenches around the outside walls. No especial thought was given to the matter until a day or so later it was observed that the center portion of the structure was gradually rising and that the area affected was constantly extending in a lateral direction around the point of initial movement. Ultimately, an area 100 ft. or more in diameter was affected with a maximum rise at the center of 3 ft. Holes were then drilled through the floor of the basin allowing the water pressure to relieve itself by flowing into the interior of the structure. Thereupon the raised portion gradually settled back into its former position without any damage other than a few slight cracks in the concrete.

Upon investigation it was learned that during the preliminary stages of construction a shallow sump had been excavated at this particular location and lateral drains had been made in the clay to lead into this sump any water that might collect. Prior to placing the concrete floor the sump and open drains had been filled with gravel, thus furnishing a blind drain through which water was admitted from the pools pocketed behind the high walls of the basin, thus permitting its uplifting influence to exert itself.

According to eye-witnesses the upward motion of the structure was very slow and as it increased the affected area widened. This would seem to indicate that the water pressure first forced itself under a sufficient area adjacent to the sump to cause a very slight upward motion which, in turn, raised the adjacent floor-slab sufficiently to enlarge the area of pressure application and thereby the movement became progressive.

Whereas it is customary to design floor-slabs of sufficient strength to permit them to transmit the upward pressure to column and wall supports, it is sometimes found to be economical to anchor the slabs into the soil by means of piles or caissons.

About 1921 a 1 000 000-gal. crude-oil reservoir was constructed in South Boston immediately adjacent to the harbor and at a depth considerably below high-water level. As there was no earth covering over the roof of the tank it was absolutely essential to prevent flotation by establishing some form of anchorage. In this instance the underlying soil was clay and a series of concrete caissons were sunk to a considerable depth below sub-grade. They were belled out at their bases while vertical reinforcing rods were embedded for the full length of the caissons and bonded into the floor of the tank. In this manner the necessary resistance against uplift was provided. The same method has frequently been used by the writer to provided intermediate reac-

tions against uplift in floor spans of considerable extent so as to permit of a more economical type of floor-slab design.

The whole subject of hydrostatic uplift has been an extremely troublesome one for many years, and it is a source of gratification to the writer to have so authoritative an expression regarding its influence as that presented by the author.

E. L. CHANDLER,* M. AM. SOC. C. E. (by letter).—The large number of experiments conducted, the very considerable range in the magnitude of the tests, and the painstaking methods followed by the author in assembling the data for this paper, serve to render his conclusions authoritative in treating a subject that heretofore has brought forth much diversity of opinion. Proper allowance for upward pressure acting on a structure founded on pervious material is a feature of design that has been poorly understood, and the information set forth in this paper is most welcome.

The total pressure developed under any structure depends, of course, on two factors: First, the effective area over which the pressure acts; and, second, the intensity of pressure. As for the first, the author's experiments seem to prove that, for a pervious soil, the area subjected to pressure is so nearly 100% of the total area that it would be unwise to assume any lesser value.

The writer has been interested in experiments to determine values for the second factor, by means of apparatus installed in dams during construction. The best data obtained were in connection with the Pinhook Dam at Maquoketa, Iowa. An account of the work done there, with an analysis of the results, has already been given in a discussion by the writer,† and details will not be repeated here. However, it is believed that the subject matter of that discussion may properly be considered in connection with Mr. Parsons' paper.

From a study of those experiments, the writer concludes that, under a dam on a pervious foundation, uplift due to the full hydrostatic head developed should be assumed at the up-stream edge, and that uplift at the down-stream edge should be taken as equal to tail-water if the apron is submerged, or to zero in the absence of submergence. At any intermediate point the uplift should be determined in accordance with the "line of creep" theory, the pressure diminishing uniformly from the up-stream to the down-stream edge in proportion to the total distance between the two edges when measured along the line of contact between the under side of the structure and the underlying material. Such measurements should take into account all cut-off walls and should follow down the up-stream face of the cut-off and then upward along the down-stream side.

It would seem, therefore, that for structures of the type under consideration, the practice of assuming uplift equal to only a fraction of the full up-stream head, or of considering the upward pressure to act on only a part of the base area, is apt to lead to unstable designs. Although it is true that the pressure may not quite reach 100% of the head, and that the effective

* With Price Bros. Co., Dayton, Ohio.

† See his discussion of the paper by Julian Hinds, M. Am. Soc. C. E., entitled "Upward Pressures Under Dams: Experiments by the U. S. Bureau of Reclamation," in this volume of *Transactions*.

area for uplift may not be quite 100% of the total base area, it appears unsafe, in the light of information given, to proceed on any lesser assumptions. This would apply only in a modified form for structures on semi-pervious materials, whereas the question of pressures acting on structures on rock must be approached in a different manner.

MARTIN J. MCPIKE,* ASSOC. M. AM. SOC. C. E.—Previous to the construction of Dry Dock No. 4 at the U. S. Navy Yard, Brooklyn, N. Y. (1908-12), some difference of opinion was found regarding the amount of hydrostatic uplift which would have to be considered in the analysis of the dry-dock design. As the hydrostatic uplift, if considered, would be a factor of prime importance and an item of some monetary significance, the question of its amount was given quite extensive thought.

Frederick R. Harris, M. Am. Soc. C. E., was at the time in charge of this project, and his experience on structures in water-bearing soil had led him to the opinion that the full hydrostatic head would have to be considered.

In view of the varied opinions expressed at that time and in order to have something tangible on which to base the design of future structures of this type, Admiral Harris had designed and placed at several positions in the floor of this dock, pressure indicators, or, as they were called, dynamometers, of the type shown in Fig. 12.

These pressure indicators were of bronze and consisted of an extra heavy pipe, 4 in. in diameter and approximately 5 ft. 8 in. long, connected to castings at the upper and lower ends. The lower casting was bell-shaped and had a diaphragm of lead, 12 in. in diameter, weighing 5 lb. per sq. ft., attached to its bottom by a bolted ring. Bearing against the upper side of this diaphragm and capable of being backed away from the face thereof by a capstan screw extending to, and operated at, the extreme upper end of the dynamometer, was a platen plate with a vertical and horizontal movement that was limited by three lugs into which set-screws, threaded into the bottom casting proper, projected.

The top casting contained a stuffing-box or gland, to prevent leakage of the pressure-transferring fluid with which the dynamometer was filled, and formed a connection for the pressure-reading gauge and a pipe for filling the dynamometer.

When it was desired to obtain a reading on any particular one of the dynamometer gauges, the capstan screw was turned, thereby withdrawing the platen plate from the face of the diaphragm and permitting it to become operative. This, in turn, through the medium of the pressure-transferring liquid (in these particular dynamometers, a light-grade oil), indicated a pressure on the gauge.

As regards location in the completed structure, the dynamometers were cast in the concrete floor of the dry dock during its construction, the lead diaphragm being placed in line with the bottom line of the dock floor, the main body of the dynamometer projecting up through the floor, about 8 ft., with the top ending in a pocket just below the upper level of the finished concrete.

* Assoc. Civ. Engr., U. S. Navy Yard, Brooklyn, N. Y.

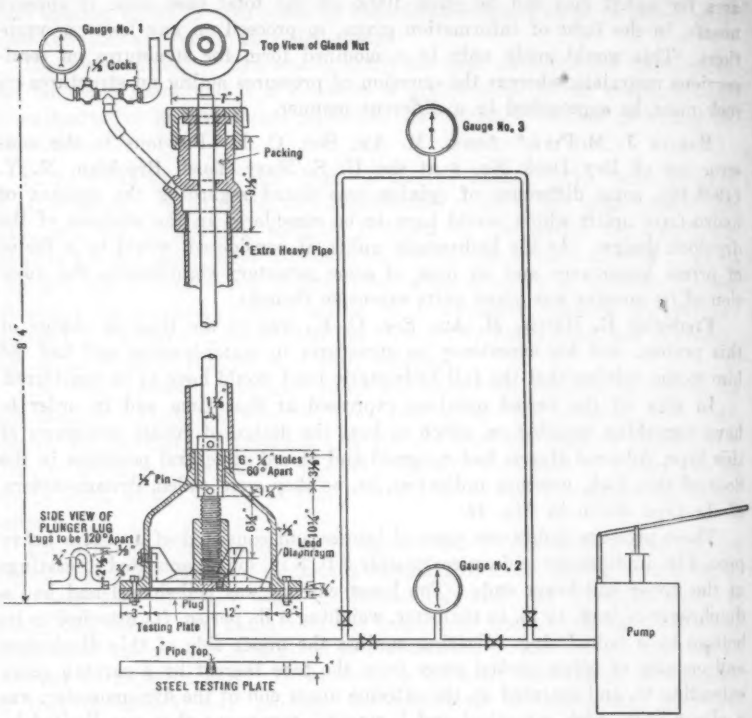

FIG. 12.—PRESSURE INDICATOR USED AT DRY DOCK NO. 4, NEW YORK NAVY YARD.

TABLE 9.—TEST NO. 4 WITH DYNAMOMETER NO. 1.*

Run No.	Pressure Increasing, in Pounds.						Pressure Decreasing, in Pounds.				
	5	10	15	20	25	30	25	20	15	10	5
1	4.0	8.0	13.0	18.5	24.0	29.0	24.5	19.0	14.0	9.5	5.0
2	5.0	9.5	14.0	19.0	24.0	28.5	23.5	19.0	14.0	10.0	5.0
3	5.0	9.5	14.0	18.5	24.0	28.5	24.0	19.0	14.0	10.0	4.5
4	4.5	9.0	14.0	18.5	24.0	29.0	24.0	19.0	14.0	9.5	4.5
5	4.5	9.5	14.0	18.5	24.0	29.0	24.0	19.0	14.0	10.0	4.5
6	4.5	9.0	14.0	19.0	24.0	29.0	24.0	19.0	14.0	9.5	4.5
7	4.5	9.0	14.0	18.5	24.0	29.0	24.0	19.0	13.5	10.0	4.5
8	4.5	9.5	14.0	18.5	24.0	29.0	24.0	19.0	14.0	10.0	4.5
9	4.5	9.0	14.0	18.5	24.0	29.0	24.0	19.0	14.0	9.5	4.5
10	4.5	9.5	14.0	19.0	24.0	29.0	24.0	19.0	14.0	9.5	4.5
Average.	4.55	9.15	13.90	18.65	24.0	28.90	24.0	19.0	13.95	9.75	4.60

* Standard gauge at top of loop; commercial gauge on dynamometer; lead diaphragm.

TABLE 10.—AVERAGES OF READINGS ON DYNAMOMETER GAUGE WITH VARYING PRESSURES OF PUMP GAUGE.

Test No.	Dynamometer No.	Diaphragm material.	Location of standard gauge.	Pressure Increasing, in Pounds.						Pressure Decreasing, in Pounds.				
				5	10	15	20	25	30	25	20	15	10	5
1	3	Lead, 5 lb.	Bottom of loop	3.25	7.80	12.65	17.00	22.05	27.00	22.87	17.94	13.69	9.19	4.00
2	2	" "	" " "	1.15	6.75	11.30	15.40	20.50	25.35	21.30	16.05	12.05	7.25	1.80
3	2	" "	Top of loop	4.30	8.80	13.55	18.10	23.10	28.10	23.20	18.25	13.60	9.65	4.70
4	1	" "	" " "	4.55	9.15	13.90	18.65	24.00	28.90	24.00	19.00	13.95	9.75	4.60
5	1	" "	On dynamometer	5.00	10.00	15.00	20.00	25.00	30.00	25.00	20.00	15.00	9.95	5.00
6	4	" "	Top of loop	4.15	9.35	14.10	18.20	23.75	28.20	23.95	18.40	13.98	9.50	4.50
7	4	" "	On dynamometer	4.70	9.45	14.45	19.35	24.15	29.05	24.20	19.15	14.45	9.15	4.60
8	4	No. 27 B. w. g. copper	Top of loop	4.40	9.53	14.45	19.40	24.30	29.15	24.38	19.68	14.75	9.98	5.00
9	4	" " "	On dynamometer	4.78	9.78	15.00	20.00	25.00	30.00	25.10	20.05	15.20	9.90	4.98
10	1	" " "	Top of loop	4.98	9.98	14.80	19.78	24.65	29.58	24.48	19.58	14.55	10.00	4.98
11	1	" " "	On dynamometer	4.73	9.83	15.05	20.08	25.08	30.00	25.18	20.00	15.23	9.90	4.93

In order that the readings on the gauges at the top level of the floor would be truly representative of the pressures encountered at the diaphragm level, approximately 8 ft. below, it was necessary to adjust the gauges used in the final installation to take care of this difference in head; hence, a series of tests was run on the dynamometers to determine as accurately as possible the "instrument factor" of each, and what loss, if any, would have to be considered in the transmission of the pressure from this lower level to the upper level.

The tests consisted in applying a variable pressure on the bottom of the diaphragm by placing a plate (see Fig. 12) below it, into which was tapped a 1-in. pipe connected to a hand-operated booster pump at the other end. The diagram gives a general idea of the arrangement of pump, gauges, etc., for the tests.

A standard gauge was used in conjunction with a commercial gauge, one being located on the pump (Gauges Nos. 2 and 3), the other being located on the dynamometer (Gauge No. 1), and at every 5-lb. increment from 0 lb. to 30 lb. and every 5-lb. decrement from 30 lb. to 0 lb. indicated on the booster-pump gauge, an observation was taken on the dynamometer gauge.

Each test consisted of ten individual runs, the readings on the dynamometer gauge were recorded, and the averages of each test were used for the final graduation of the dynamometer gauges.

Both lead (5 lb. per sq. ft.) and copper (No. 27 B.w.g.) diaphragms were used in the tests, but lead diaphragms were selected for the completed structure, owing to the most consistent results being obtained therefrom in the tests.

A typical test run is indicated in Table 9. In order that the final results should be consistent under all conditions likely to be encountered in the field, a great deal of care was exercised in running the tests, the position of the gauges varying for the different tests and indicated in Fig. 12 and Table 10. The averages of all tests run, as well as the variation in readings in the dynamometer gauge under the different conditions of gauge location and diaphragm material, are also given in Table 10.

After the completion of the dry dock in 1912, observations were taken on these dynamometer gauges four times daily for a period of several months, and their readings, plotted in conjunction with tidal observations, indicated practically 100% hydrostatic uplift fluctuating with tidal changes.

After the dynamometers had served the purpose for which they were installed, the gauges were removed and the pockets filled with concrete. In 1918, two of the dynamometers were uncovered, and observations taken thereon were consistent with the results previously obtained. On June 15, 1928, the speaker again had the opportunity of testing the effectiveness of two of these dynamometers, and they indicated a pressure of exactly the same intensity as they did in 1918.

JACOB FELD,* ASSOC. M. AM. SOC. C. E. (by letter).—This paper should settle, once and for all, the disputed question of the transmission of hydrostatic pressure through pervious soils. The contentions of some engineers

* Cons. Engr., New York, N. Y.

that the area of a wall covered by grains of soil was not affected by the pressure of the water surrounding such grains had no theoretical basis and was never previously tested. The sole basis of the contention was the fact that dams and other structures built on such an assumption had not failed. These experiments prove definitely that the hydrostatic pressure against a surface cannot be decreased by placing a granular material against that surface.

The method of drawing conclusions from the first set of tests is open to criticism. Table 1, giving averages of friction tests, in pounds per square inch, on the side of the can in contact with sand, is purely an arbitrary set of figures which cannot be applied to any other case. A factor which will influence these values is the height of water overlying the top of the sand. The friction of the can embedded in sand, occurring as it does at the point of motion, is a function of the pressure of the sand against the can. The amount of this pressure depends on the depth and the nature of the sand, and the latter depends on the water overlying it. As a result the use of Fig. 3 to determine the percentage of base area on which hydrostatic uplift acts, gives results which are not quite consistent. In analyzing the data in Table 2 it is interesting to note that the average percentage of effective area on which hydrostatic uplift acts is 95.7 for the metal can 15.09 in. in diameter, 88.5 for the 9.22-in. can, and 86.7 for the 5.53-in. can. This is for the condition of wet sand packed in the container. The other set of experiments in the first part, enumerated in Table 3, shows a number of low values for the 9.22-in. can when the sand is packed dry in the container with, later, an addition of water. This can had a plaster-coated bottom and it is highly probable that there was a considerable quantity of air trapped on the surface of the bottom. However, the data from the other two sets of tests are so conclusive that there is no doubt of the validity of the results.

The author states that he found no appreciable difference in friction due to the materials of which the cans were made, namely, glass, tin, or galvanized iron. The writer believes that in all work on sand and similar soils the same conclusions can be drawn, because the friction between materials like glass, tin, galvanized iron, or wood and sand, is practically the same as the friction of sand against sand. As a result, the true frictional value measured is practically that of sand against sand since a thin layer of the filling material adheres to the test surface and true motion is not between the filling material and test surface, but is a shearing deformation in the filling material itself. Similar results and conclusions were obtained by the writer in his experimental work on the determination of the lateral pressure of soil against walls made of wood, glass, and sheet metal.* Similar results were also obtained by Müller-Breslau in 1906 in his experimental work in earth pressures. Certainly the friction between sand and the ordinary concrete or stone used for dams is greater than the friction between sand and such test materials, and the conclusion therefore applies to all actual conditions.

In reference to the tests with clay, the lack of results is explained in the paper by the fact that the clay became liquid in nature. Under such condi-

* *Transactions*, Am. Soc. C. E., Vol. LXXXVI (1923), pp. 1472, 1476, and 1477.

tions the pressure exerted is given by the formulas in hydrostatics and the same conclusions given for sand then apply to clay. That liquid materials, such as a concrete mix, obey the laws of hydrostatics has been proved by the experimental work of Messrs. F. R. Schunk* and E. B. Smith.†

Referring to the experiments of the first set, the amount of friction measured is an indication of the total lateral force acting against the submerged can. The ratio between the vertical force (resistance measured in the tests) to the horizontal pressure of the saturated sand against the can must equal the coefficient of friction between saturated sand and the material of which the can was made. This coefficient can be experimentally determined and is quite constant for all the usual materials, even for such unlike materials as wood, tin, and glass. The results are then directly applicable to the design of walls, quays, or docks acted upon by saturated soils and also to dams against which silt has accumulated to considerable heights. The writer would like to know whether any designer of dams has investigated the change in pressures sustained by a dam because of silt accumulation in the reservoir.

J. Albert Holmes,‡ M. Am. Soc. C. E. (by letter).—The author, after careful experimentation, prolonged study, and weighing of evidence, has placed before the Engineering Profession the data in a problem which he has ample reason to believe embodies a correct interpretation of the facts. Still, considering the weight of the evidence in earlier experiments,§ he probably hesitated long and re-weighed the evidence with great care before reaching so divergent a conclusion.

Mr. Parsons states that his research was made "to determine the hydrostatic uplift on the base of a structure, founded on a pervious soil, when there is no loss of head due to velocity of flow". This is static head, acting by mere weight without motion. There was some replacement by water when pistons were actuated in the apparatus used by Mr. Parsons. The pressure, when applied to the water above the sand, must necessarily build up throughout the whole body of the sand, not only to the depth of the piston end, but to the bottom of the chamber, before actuating the piston.

The soil particles in a bed of pervious material must have appreciable size, otherwise the material will be impervious. Mr. Parsons refines the matter minutely when he states that the resultant of pressure of a particle of soil in contact with the base of a structure is upward. This is so, especially when the contact is but a point, since it could not be much more than a point. Rather than any considerable part of the base of a structure being in contact with the pervious soil particles, it would seem more likely to be in contact with water.

The writer's interest in the problem of hydrostatic uplift relates to dams, and more particularly to earth dams where there is flow through the soil and a hydraulic gradient.

* *Professional Memoirs*, U. S. Corps of Engrs., Vol. 1, No. 3, 1909, pp. 247-260.
† *Proceedings*, Am. Concrete Inst., 1920.
‡ Hydr. Engr., Pearse, Greeley & Hansen, Chicago, Ill.
§ *Transactions*, Am. Soc. C. E., Vol. LXX (December, 1910), p. 364.

In all hydraulic experiments with soils, the presence of air is a disturbing factor affecting flow. Air is also present in the ground under dams and, of course, affects seepage rates there. An instance is recalled where air escaping upward through the water from an isolated mound of original ground, kept ice from forming just above on the core pool of an earth dam during a cold winter.

E. W. LANE,* M. AM. SOC. C. E. (by letter).—The profession is greatly indebted to the author for giving to it the results of such thorough experimentation on a subject which is of great importance, but of which so little is known. The application of the experiments is not confined to dams, because, judging from the number of cases which have come to the writer's attention of buried tanks, cisterns, and even septic tanks, being lifted from their positions by upward pressure of the water acting through the surrounding soil, the costly accidents resulting from a neglect of this pressure must be legion.

For many years the writer has been interested in the effect of upward pressure on dams on porous foundations. Some information was available on the unit pressure beneath dams, but the writer could find only one attempt to determine the area on which the pressure acted.† The writer did not regard these experiments sufficiently conclusive, and, therefore, made a few of his own. The method used was very similar to that used by Mr. Parsons in his low-head tests. The bottom of a large vessel (Fig. 1) was filled with sand. A conical depression, the size of a smaller vessel, was scraped in the sand and filled with paraffin, which had been heated just above melting temperature, in order that it would make a contact with the sand similar to concrete poured on a gravel base, without running deeply into the voids. The depression was made conical to prevent the entraining of air beneath the smaller vessel. While the paraffin was still liquid, a smaller vessel with vertical sides was set into it, and the paraffin allowed to harden in contact with the inner vessel. This vessel was then filled with water nearly to the top, and water was added slowly to the outer vessel until the inner one floated.

The results were not developed quantitatively, since the height of water in the outer vessel (with respect to the inner one) which was necessary to lift the inner vessel, was very little more than that necessary to float the inner one after being lifted free from the sand. It was evident, therefore, that the pressure through the sand acted on practically the entire area of the base of the inner vessel. The slight difference in height in the two cases might easily be due to friction in breaking the contact of the paraffin and the sand adhering to it and the main body of sand. This difference also includes the discrepancy, if any, due to a time element in the transmission of pressure through the sand as the water was poured into the outer vessel. If the writer were to repeat the experiments, he would adopt Mr. Parsons' method, and siphon the water from the inner vessel, instead of adding it to the outer one, as the former method eliminates all question regarding a possible time of transmission of pressure through the sand. However, error from this source could

* Engr., U. S. Bureau of Reclamation, Denver, Colo.

† "Pressure, Resistance, and Stability of Earth," by J. C. Meem, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXX (1910), p. 352.

not have been appreciable, since both it and the friction in breaking contact with the bottom are included in the very slight amount the inner vessel was lifted when the contact with the sand bed was broken.

H. DE B. PARSONS,* M. AM. SOC. C. E. (by letter).—A full and frank discussion increases knowledge, and this is especially true when the subject is one of experimentation and research. The writer, therefore, thanks those who have criticized his paper. Some of the discussions require no comment, and consideration of the others will be as brief as the facts will permit.

Mr. Meem infers that the pistons or contact rods were not pressed or forced into the sand beds so as to secure full contact, but were simply rested on the sand. The experimental rods were forced down firmly, and their unit loading was as heavy as that under many buildings. The cans were pressed down by hand to compress the sand beneath their bases and establish definite areas of contact. In some of the tests made at Rensselaer Polytechnic Institute, the apparatus was allowed to stand, heavily loaded, for 18 to 48 hours, but the results were substantially the same as when the tests were made (with saturated sand) shortly after arranging the apparatus. Mr. Meem's experiments are interesting, but seem, to the writer, to embody dynamic rather than strictly hydrostatic conditions. Professor Terzaghi's discussion should be read with Mr. Meem's contribution.

Mr. Gallogly stated that the apparatus seemed to illustrate the conditions surrounding a river caisson only, and that the tests do not seem to cover all the cases of foundations constructed in pervious soils below ground-water level. The writer would like to call attention to the fact that his tests were made to determine whether the hydrostatic uplift acted on the full area of the foundation base, or only a part, called the "effective area". This being so, it would seem that the results apply to any kind of structure founded below water level.

The discussion by Professor Terzaghi is both instructive and constructive. The writer was unaware of any tests published in the English language before his paper, and he undertook this research because repeated demands were being made for information by American engineers.

He found, at the beginning, the difficulty of excluding the air content from the sand; and it was one of the causes which delayed the final tests for about nine months, as the air produced erratic results. A test was made by inserting a rubber tube in the sand bed, one end being against the base of a can and the other joining a glass tube having the open end above the water level. The water level in the containing vessel was then raised, and the time noted for the water in the glass tube to come to the same level. It took many hours for the water in the tube to rise, and it frequently did not quite reach the level of the water in the container. The writer and his aides recognized this as air, and the technique was changed to saturate or try to saturate the sand.

The tests made at Stevens Institute of Technology for gravel, confined sand, and sand not confined, appeared to show that the depth of material below

* Prof. Emeritus, Rensselaer Polytechnic Inst.; Cons. Engr., New York, N. Y.

the base of the contact rod had an influence on the effective area, as shown in Fig. 5. This result appears to have been substantiated by the high-head tests at Rensselaer Polytechnic Institute.

All tests were repeated under similar conditions, and the results averaged. (See Tables 1 to 7.) The results with the gravel confirmed those with the sand, as regards the influence of the depth of the material. The gravel was put in the container with water, and the rod pounded down, like a battering ram, on the gravel, to compact it before completely filling the container with water. On account of the relatively large voids between the pebbles, and the manipulation used, it would seem that air was practically excluded. In view of the cases cited by Professor Terzaghi, the writer can reply only by saying that the conclusion, regarding the depth of material required under the base to get substantially 100% uplift, was in accordance with the results as found.

An explanation of the phenomenon may be somewhat as follows: Under the load applied by the contact rod, or by that of a base of an ordinary structure, there can be no appreciable deformation of the sand or gravel particles, although the particles are firmly pressed together in close contact. The weight is supported on the contact areas between the base and the top layer of the particles. There was no hydrostatic head on these areas, as the contact excluded the water. The top layer (that is, the one next to the base) was supported by other layers of particles, between which there were contact areas on which the hydrostatic head also did not act. If the particles had been arranged in perfect alignment, uniform in size, and of oval form, as in Fig. 9, the effective area would be less than the area of the base by an infinitely small amount, corresponding to the elastic deformation of the particles, as stated by Professor Terzaghi; if, with a similar arrangement, the contact areas of the particles were flat and of measurable extent, the effective area would be less than the total area of the base, since the head would act only on that part between the contact areas, no matter how deep the sand was under the base.

The fact is that the particles varied widely in size and shape, and were in every conceivable position relative to one another; and the contact areas on the top and the bottom of a particle would not be always in the same vertical line. The head, acting on the top half of a particle exposed to the water, would give a downward resultant, to which force would be added the weight of the particle. Likewise, the head acting on the bottom half would give an upward resultant, the head being greater due to the vertical thickness of the particle. This increase in head would be extremely small for one particle, but would be measurable as the layers increased. It would seem reasonable that some depth of material would be required before the exposed areas on the bottoms of the particles, with their hydrostatic heads, would be sufficient to overcome the exposed areas on the tops with their heads, so as to transmit the uplift to the base through the points of contact. While appreciating Professor Terzaghi's criticism, it is conceivable that an analytical treatment might not be as accurate as the results of high-head tests.

Mr. Feld is correct instating that the sand friction tests, given in Table 1 and shown graphically in Fig. 3, are not to be applied to other work. The condition of the sand is described in the paper. The water levels for the can tests (low-head tests) were within a few inches of the top edges of the cans.

Mr. Holmes states that in the high-head tests there was "some replacement by water when the pistons were actuated". The pistons were stationary during the test. When a piston commenced to move, the test was over.

The water pressure in the sand or gravel body built up as the head was increased, but without movement, as the water can be considered as incompressible under the pressures used. The only chance for "flow" was entrapped air, and this has been discussed in the writer's reply to Professor Terzaghi.

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Paper No. 1714

CONTINUOUS BEAMS OVER THREE SPANS*

By I. Oesterblom,† M. Am. Soc. C. E.

With Discussion by Messrs. Frank S. Bailey, Clyde T. Morris, A. Lincoln Hyde, Harold E. Wessman, Robert M. Wilder, W. F. Way, H. M. Hadley, T. F. Hickerson, John I. Parcel, R. A. Caughey, John B. Snethlage, T. D. Mylrea, and I. Oesterblom.

Synopsis

The matters of accessibility, natural light, and fresh air combine to make three-span construction more prevalent than any other series of multiple spans for buildings.

In the colder, and even in the temperate, climates, office buildings, schools, hospitals, hotels, clubs, etc., generally provide for corridors in the center with rooms on either side. In the tropics the same classes of buildings, and also the dwellings, have rooms in the center with verandas on either side. Thus, the problems of two long side spans with a short center span, or two short side spans with a long center span, are constantly offered to the engineer for solution.

For continuous construction, particularly prevalent on account of the increasing use of reinforced concrete, the proper solution of the moment equations is a most tedious task. It has come to be a very frequent custom, therefore, to neglect such important matters as the effect on moment factors of variability in spans, loads, and sections.

This erroneous custom is sometimes very serious in its consequences. It reaches to all divisions of engineering—design, control, and execution; consultants, municipal authorities, and manufacturers of specialties, all are affected. Even engineering committees and technical schools seem to have avoided the problem or condoned the errors.

* This paper was published in March, 1928, *Proceedings*.
† Structural Engr., St. Louis, Mo.

In textbooks from Continental Europe, the problem is usually covered by the evolution of formulas which may be used in good faith and with good results, although they are generally tedious to apply.* Tables to facilitate the use of the formulas are generally missing, or are so elaborate as to be of no use for the most common problems of variable spans in commercial engineering work. The economic significance of the variables affecting the moment factors is also overlooked.

This paper aims to provide the necessary tables showing moment factors for all the most common conditions, so that the moments may be found with the expenditure of less effort and time. It also shows the significance of the variables and the seriousness of the usual conventional errors. By the use of correct methods, safety and economy of materials are obtained simultaneously.

The Three-Moment Equation

The "three-moment equation" is presented in many different forms, one of which is the following:

$$(M_1 + 2\,M_2)\,\frac{l_{12}}{I_{12}} + (2\,M_2 + M_3)\,\frac{l_{23}}{I_{23}} = -\,\frac{6\,A_{21}}{I_{12}} - \frac{6\,A_{23}}{I_{23}} \ldots\ldots (1)$$

The symbols have the usual significance:

M_1, M_2, etc. = bending moments (at supports).
l_{12}, l_{23}, etc. = spans.
I_{12}, I_{23}, etc. = moments of inertia.
A_{21}, A_{23}, etc. = moment area reaction for simple span conditions.†

The proper meanings of the subscript indices in relation to supports and spans are shown on Fig. 1, according to the following system:

(1) Supports are numbered 1, 2, 3, and 4.
(2) Spans are numbered 12 between Supports 1 and 2; 23 between Supports 2 and 3; etc.
(3) Reactions are numbered as the supports.
(4) Unit loads and moments of inertia are numbered as the spans.
(5) Concentrated loads are numbered as the spans, with subscript numbers for each span to indicate locations, as 121, 122, 123, etc., for loads on Span 12.
(6) The area moment reactions take the first subscript number from the support and the second from the adjacent support on the contributory side. For example, A_{32} is the reaction at Support 3 due to the moment area between Supports 3 and 2.

The form of Equation (1) is "universal" in so far as it takes into account a different section for each span of the beam and any condition of loading. It is not completely universal, however, in so far as it neglects the effects of settlement of supports, or changes due to temperature or similar causes. For purposes of ordinary construction the form is sufficiently comprehensive, as variations in span, section, and load are the only factors that ordinarily affect the stresses.

* See, especially, formulas in "Beton Kalender," pub. by Wilhelm Ernst & Sohn, Berlin, Germany, which contains some of the equations derived by the writer.

† That is, the reaction of the area under the moment curve considered as a load, the moment curve being obtained for simple span conditions.

In the left side of Equation (1) are all the "moment terms"; in the right side are the "load terms". The reasons for these designations are obvious.

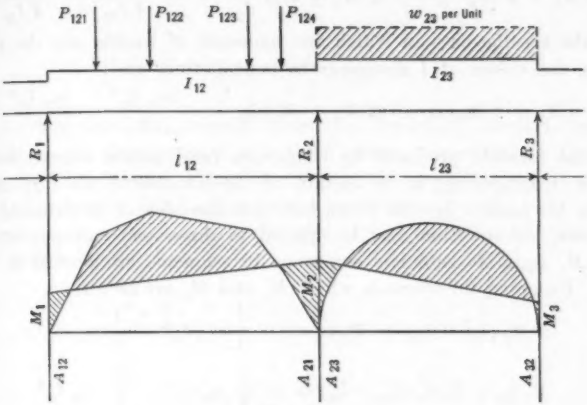

FIG. 1.

Methods of Solution.—For a two-span construction with no end restraints, the end moments, therefore, being equal to zero, the moment at the center support can be found by means of one moment equation only. For each additional span under similar conditions as regards end restraint, one additional equation will be required. The solution of several equations of this type is an elaborate task. Determinants will have to be resorted to for three or more equations. This simplifies the task of solution, but it still requires a great deal of time. For several spans it is customary to use graphic methods, by means of which results can be obtained quickly and accurately. The methods now in use take into account all the usual variables—spans, loads, and moments of inertia.

The graphic method may be the only practical method for five spans or more—even for four spans, it may prove to be the best. For three spans, however, and especially for commercial "repeat work" involving merely minor variations, it would be desirable to have a method developed in detail with moment factors available for immediate use. It is the object of this paper to provide these moment factors.

For commercial work, uniform loads or concentrated loads at equal spacing are especially significant. Other cases may be considered as unusual and for them no moment factors need be developed; they should be treated individually. Moments due to equi-distant concentrated loads are so nearly equal to uniform loads that the same moment factors would apply. Consider, therefore, the significance of the area-moment reactions and the need of making the "load terms" more definite.

CLAPEYRON'S THEOREM

The moment area for a uniform load is parabolic, and, therefore,

$$2A = \frac{2}{3} \times \frac{w l^2}{8} \times l = \frac{w l^3}{12} \quad \ldots\ldots\ldots\ldots\ldots\ldots (2)$$

Hence, the three-moment equation in more definite form would be,

$$(M_1 + 2\,M_2)\,\frac{l_{12}}{I_{12}} + (2\,M_2 + M_3)\,\frac{l_{23}}{I_{23}} = -\,\frac{w_{12}\,l_{12}^{\,3}}{4\,I_{12}} - \frac{w_{23}\,l_{23}^{\,3}}{4\,I_{23}} \quad \ldots\ldots(3)$$

For the specific case in which the moments of inertia are the same for all spans, the values of I disappear by cancellation, and:

$$(M_1 + 2\,M_2)\,l_{12} + (2\,M_2 + M_3)\,l_{23} = -\,\frac{w_{12}\,l_{12}^{\,3}}{4} - \frac{w_{23}\,l_{23}^{\,3}}{4} \quad \ldots\ldots(4)$$

This is the formula produced by Clapeyron (and almost simultaneously by two other investigators) in the middle of the Nineteenth Century and since known by his name. In this form, but with the idea of introducing amendments later, the equation may be applied to three-span construction. With M_1 and M_4 equal to zero two equations in sequence are needed to find M_2 and M_3. Dropping the terms in which M_1 and M_4 are factors,

$$2\,M_2\,(l_{12} + l_{23}) + M_3\,l_{23} = -\,\frac{1}{4}\,w_{12}\,l_{12}^{\,3} - \frac{1}{4}\,w_{23}\,l_{23}^{\,3} \quad \ldots\ldots(5)$$

$$M_2\,l_{23} + 2\,M_3\,(l_{23} + l_{34}) = -\,\frac{1}{4}\,w_{23}\,l_{23}^{\,3} - \frac{1}{4}\,w_{34}\,l_{34}^{\,3} \quad \ldots\ldots(6)$$

Support Moments.—For further operations a symbol, K, for an oft-occurring expression is introduced as follows:

$$K = 4\,(l_{12}\,l_{23} + l_{12}\,l_{34} + l_{23}\,l_{34}) + 3\,l_{23}^{\,2} \quad \ldots\ldots(7)$$

Solving for $M_3\,l_{23}$ in Equation (5) and introducing the values of M_3 in Equation (6):

$$M_3\,l_{23} = -\,\frac{1}{4}\,w_{12}\,l_{12}^{\,3} - \frac{1}{4}\,w_{23}\,l_{23}^{\,3} - 2\,M_2\,(l_{12} + l_{23}) \quad \ldots\ldots(8)$$

$$-\,K\,M_2 = \frac{1}{2}\,w_{12}\,l_{12}^{\,3}\,(l_{23} + l_{34}) + \frac{1}{2}\,w_{23}\,l_{23}^{\,3}\left(\frac{1}{2}\,l_{23} + l_{34}\right) - \frac{1}{4}\,w_{34}\,l_{34}^{\,3}\,l_{23} \quad \ldots\ldots(9)$$

Equation (9) now will give all the support moments, M_2, and after these are found all the support moments, M_3.*

First, considering Span 12 only loaded with w_{23} (w_{34}, therefore, being equal to zero),

$$-\,K\,M_2 = \frac{1}{2}\,w_{12}\,l_{12}^{\,3}\,(l_{23} + l_{34})$$

and,

$$M_2 = -\,\frac{w_{12}\,l_{12}^{\,3}}{2\,K}\,(l_{23} + l_{34}) \quad \ldots\ldots(10)$$

For the second span loaded, and with w_{12} and w_{34} equal to zero, in like manner,

$$-\,K\,M_2 = \frac{1}{2}\,w_{23}\,l_{23}^{\,3}\left(\frac{1}{2}\,l_{23} + l_{34}\right)$$

and,

$$M_2 = -\,\frac{w_{23}\,l_{23}^{\,3}}{4\,K}\,(l_{23} + 2\,l_{34}) \quad \ldots\ldots(11)$$

* The term, "support moment," refers to the moment at a support as distinguished from "span moment," within the span itself.

Finally, with Span 34 only loaded, and with w_{12} and w_{23} equal to zero,

$$-K M_2 = -\frac{1}{4} w_{34} l_{34}^3 l_{23}$$

and,

$$M_2 = \frac{w_{34} l_{34}^3}{4 K} l_{23} \quad \ldots\ldots (12)$$

Thus, have been found the moment effects for the second support due to uniform loads on the first, second, and third spans, respectively. The expressions for M_2 being found, Equation (8) may be used, together with Equations (10), (11), and (12), to find similar expressions for M_3.

Setting w_{12}, w_{23}, and w_{34} equal to zero in like manner, the equations for Span 12 loaded will be,

$$K M_3 l_{23} = -\frac{1}{4} K w_{12} l_{12}^3 + w_{12} l_{12}^3 (l_{12} + l_{23}) (l_{23} + l_{34}) = \frac{1}{4} w_{12} l_{12}^3 l_{23}^2$$

and,

$$M_3 = +\frac{w_{12} l_{12}^3}{4 K} l_{23} \quad \ldots\ldots (13)$$

Similarly, for Span 23 loaded:

$$K M_3 l_{23} = -\frac{1}{4} K w_{23} l_{23}^3 + \frac{1}{2} w_{23} l_{23}^3 (l_{12} + l_{23}) (l_{23} + 2 l_{34})$$

$$= -\frac{1}{4} w_{23} l_{23}^3 (2 l_{12} l_{23} + l_{23}^2)$$

and,

$$M_3 = -\frac{w_{23} l_{23}^3}{4 K} (2 l_{12} + l_{23}) \quad \ldots\ldots (14)$$

Finally, for Span 34 loaded:

$$K M_3 l_{23} = -2 M_2 (l_{12} + l_{23}) = -\frac{1}{2} w_{34} l_{34}^3 (l_{12} + l_{23}) l_{23}$$

and,

$$M_3 = -\frac{w_{34} l_{34}^3}{2 K} (l_{12} + l_{23}) \quad \ldots\ldots (15)$$

Span Moments.—From the expressions for the support moments one may proceed to find equations for the span moments through the medium of expressions for the reactions. The relations between support moments and reactions for any one span are given by the equation,

$$R_L l_{12} = M_2 - M_1 + \frac{1}{2} w_{12} l_{12}^2 \quad \ldots\ldots (16)$$

In like manner the relations between span moments and the reactions will be,

$$M_x = M_1 + R_L x - \frac{1}{2} w_{12} x^2 \quad \ldots\ldots (17)$$

R_L indicates the reaction at the left end of the span and M_x, the span moment at a distance, x, from the left support. $M_{x\,\max.}$ is expressed by M_{12}, M_{23}, and

M_{34}, respectively. Differentiating M_x in regard to x, and setting the derivative equal to zero, gives for $M_{x\,\max.}$:

$$x = \frac{R_L}{w_{12}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (18)$$

and,

$$M_{x\,\max.} = M_1 + \frac{{R_L}^2}{2\,w_{12}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (19)$$

Solving for R_L in Equation (16) and using that value in Equation (19):

$$M_{12} = M_1 + \frac{1}{2\,w_{12}}\left(\frac{M_2}{l_{12}} - \frac{M_1}{l_{12}} + \frac{w_{12}\,l_{12}}{2}\right)^2 \quad \ldots\ldots\ldots\ldots (20)$$

Equations (18) and (19) are useful and important; but Equation (20) is the one to be used for the specific purpose of finding maximum span moments. All three equations obviously are good for any span with proper modifications of the indices for M, w, and l. Therefore, the following equations hold for Spans 23 and 34:

$$M_{23} = M_2 + \frac{1}{2\,w_{23}}\left(\frac{M_3}{l_{23}} - \frac{M_2}{l_{23}} + \frac{w_{23}\,l_{23}}{2}\right)^2 \quad \ldots\ldots\ldots\ldots (21)$$

$$M_{34} = M_3 + \frac{1}{2\,w_{34}}\left(\frac{M_4}{l_{34}} - \frac{M_3}{l_{34}} + \frac{w_{34}\,l_{34}}{2}\right)^2 \quad \ldots\ldots\ldots\ldots (22)$$

It should be noted that for the particular cases under investigation all terms containing M_1 and M_4 disappear, as these two moments are always zero.

Moment Effects from Adjacent Spans

As may be noted, Equations (20), (21), and (22) will give expressions for the maximum bending moments in the loaded spans, but obviously will not give any results for adjacent spans. This is on account of the presence of the zero load element, w, as a divisor in the second term of the equations, which makes the expressions indefinite. To find the meaning of this indefinite feature the three-moment diagrams may be considered in sequence, as shown in Fig. 2.

As previously, the supports are indicated by the numerals, 1, 2, 3, and 4; the spans by the numbers, 12, 23, and 34; and the conditions of loading by the letters, A, B, and C. In Fig. 2, A shows a diagram for Span 12 loaded; B, for Span 23; and C, for Span 34. The quantities, M_{f12}, M_{f23}, and M_{f34}, would be the moments at points of maximum effect for the loaded spans if they were freely supported and not continuous; and M_{12}, M_{23}, and M_{34}, the actual maximum moments for the loaded spans.

It is obvious from Fig. 2 that there can be no maximum span moments in the interior of spans adjacent to the loaded spans. This, therefore, is the interpretation of the indefinite feature of Equations (20), (21), and (22).

Combination of Moment Effects.—For purposes of analysis the individual diagrams are instructive. As the ultimate aim of this paper is practical, however, the results from the combination of the three are more interesting. It is not desired to know the maximum effect in the unloaded spans, but rather the effect where the combined moments will be a maximum.

In individual cases it is easy to establish the points of maximum effect for loaded spans due to the loads on these spans. Equations (16) and (18)

will give these with little effort after the negative moments have been found. After the addition of secondary effects from adjacent spans the points of maximum moment will not be quite the same. The combined moments at the two points would be substantially the same, however, as the change of curvature between the points is insignificant.

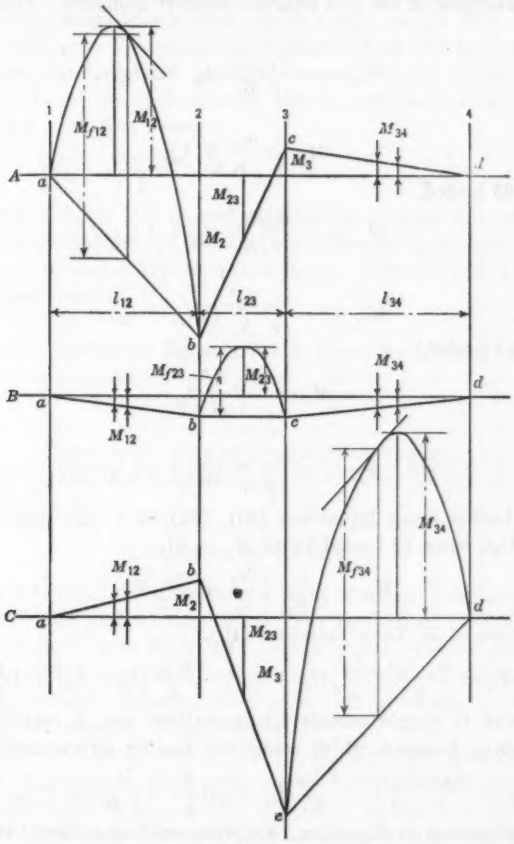

FIG. 2.

It is intended finally to establish definite moment factors for given ratios of span lengths. It would lead too far afield to establish without error the secondary moment effects from loads in adjacent spans. Instead it is necessary to introduce an approximation, by assuming that the secondary effects at the mid-spans will give results close enough to be added to the two principal moments. An inspection of Fig. 2 will show that for the totals this is substantially correct. On all the spans the lines representing the two unloaded effects are compensating, and the resultant of the two is almost hori-

zontal. In the outside spans the secondary effects are generally small, while in the center spans the mid-points in most cases of reasonable design are truly points of maximum combined effect. All these considerations combine to make the proposed approximation almost equal to definite accuracy.

On this basis the secondary mid-span moment effects can now be written directly as averages of the two adjacent support moments. Then, with Span 12 loaded,

$$M_{23} = -\frac{w_{12}\, l_{12}^{\,3}}{8\, K}\,(l_{23} + 2\, l_{34}) \qquad (23)$$

and,

$$M_{34} = \frac{w_{12}\, l_{12}^{\,3}}{8\, K}\, l_{23} \qquad (24)$$

With Span 23 loaded,

$$M_{12} = -\frac{w_{23}\, l_{23}^{\,3}}{8\, K}\,(l_{23} + 2\, l_{34}) \qquad (25)$$

and,

$$M_{34} = -\frac{w_{23}\, l_{23}^{\,3}}{8\, K}\,(2\, l_{12} + l_{23}) \qquad (26)$$

With Span 34 loaded,

$$M_{12} = \frac{w_{34}\, l_{34}^{\,3}}{8\, K}\, l_{23} \qquad (27)$$

and,

$$M_{23} = -\frac{w_{34}\, l_{34}^{\,3}}{8\, K}\,(2\, l_{12} + l_{23}) \qquad (28)$$

For the loaded spans Equations (20), (21), and (22) may now be used. Assuming, first, Span 12 loaded (with $M_1 = 0$),

$$M_{12} = \frac{1}{2\, w_{12}}\left(\frac{M_2}{l_{12}} + \frac{w_{12}\, l_{12}}{2}\right)^2$$

and with values of M_2 from Equation (10),

$$M_{12} = \frac{w_{12}\, l_{12}^{\,2}}{8}\left\{\frac{l_{12}^{\,2}}{K^2}\,(l_{23} + l_{34})^2 - \frac{2\, l_{12}}{K}\,(l_{23} + l_{34}) + 1\right\} \qquad (29)$$

This expression is simple enough for immediate use; it may be written in a different form, however, which sometimes has its advantages,

$$M_{12} = \frac{w_{12}\, l_{12}^{\,2}}{8}\left\{1 - \frac{2\, l_{12}}{K}\,(l_{23} + l_{34})\left[1 - \frac{l_{12}}{2\, K}\,(l_{23} + l_{34})\right]\right\} \qquad (30)$$

If the square bracket in Equation (30) were omitted entirely, the remainder would represent the mid-span moment, which has been suggested by some authorities as sufficiently accurate. For a loaded end span the error is considerable, as may be seen if the expression inside the omitted bracket be evaluated.

For Span 34 loaded, the span moment equations can now be written directly on account of the symmetry of the case. Substituting Indices 12 for 34 and 34 for 12 in Equations (29) and (30),

$$M_{34} = \frac{w_{34}\, l_{34}^{\,2}}{8}\left\{\frac{l_{34}^{\,2}}{K^2}\,(l_{23} + l_{12})^2 - \frac{2\, l_{34}}{K}\,(l_{23} + l_{12}) + 1\right\} \qquad (31)$$

and,

$$M_{34} = \frac{w_{34}\, l_{34}^2}{8} \left\{ 1 - \frac{2\, l_{34}}{K} (l_{23} + l_{12}) \left[1 - \frac{l_{34}}{2\, K} (l_{23} + l_{12}) \right] \right\} \ldots\ldots (32)$$

For moment effects in the center span due to loading in that span, again it is possible to write an approximation with a considerable accuracy and thereby eliminate long calculations, which would hardly add anything of practical significance. The maximum moment effect will be found so close to mid-span and the disturbing negative moment line for all reasonable span conditions will be so nearly horizontal that a mid-span expression for M_{23} is fully justified. On this basis,

$$M_{23} = \frac{w_{23}\, l_{23}^2}{8} \left\{ 1 - \frac{2\, l_{23}}{K} (l_{12} + l_{23} + l_{34}) \right\} \ldots\ldots\ldots\ldots (33)$$

To ascertain directly and immediately useful moment factors for all the most frequently occurring span and load conditions in three-span construction all the necessary formulas have now been found. Table 1 gives a summary of those expressions required.

TABLE 1.—SCHEDULE OF EQUATIONS FOR VARIOUS MOMENTS AND SPANS.

Load on:	M_{12}.	M_2.	M_{23}.	M_3.	M_{34}.
Span 12.....	Equation (30)	Equation (10)	Equation (23)	Equation (13)	Equation (24)
Span 23.....	Equation (25)	Equation (11)	Equation (33)	Equation (14)	Equation (26)
Span 34.....	Equation (27)	Equation (12)	Equation (28)	Equation (15)	Equation (32)

CONDITIONS OF SYMMETRY

Clearly it is impracticable to make a table of moment factors for all possible and reasonable span combinations. For increments of 6 in. and with a range from 8 ft. 0 in. to 30 ft. 0 in., which would cover all common cases, there would be no less than 2 025 combinations, for each of which there would have to be 15 moment factors. Neither is this necessary, as the moment factors are merely matters of span ratios.

This analysis will be limited to conditions of perfect symmetry or slight asymmetrical variations, as these cases are the only ones of frequent practical occurrence. First, let $l_{12}=l_{34}=l$ and $l_{23}=xl$, in which, x is the span ratio. Then moment factors may be established for all span ratios between 0.20 and 2.50 in increments of 0.10. Finally, it will be pointed out how these moment factors may be used to ascertain correctly moments for cases of asymmetric span relations.

For this purpose the equations enumerated in Table 1 will be used, with the span designations as noted and with the loads for the loaded spans equal to unity in proper sequence. The moment factor formulas (Table 2) are merely simplifications as outlined on the basis of the equations noted in Table 1. The table of "Moment Factors" (Table 3) is a further development for given span ratios.

TABLE 2.—MOMENT FACTOR FORMULAS FOR THREE-SPAN CONSTRUCTION.
(Two outside spans alike and equal to unity; middle span equal to x.)
$(K = 8x + 3x^2 + 4)$

Loading.	M_{12}.	M_2.	M_{23}.	M_3.	M_{34}.
Span 12 loaded.	$\frac{1}{8} - \frac{x+1}{4K}\left(1 - \frac{x+1}{2K}\right)$	$-\frac{x+1}{2K}$	$-\frac{x+2}{8K}$	$+\frac{x}{4K}$	$+\frac{x}{8K}$
Span 23 loaded.	$-\frac{2x^3 + x^4}{8K}$	$-\frac{2x^3 + x^4}{4K}$	$\frac{x^2}{8} - \frac{2x^3 + x^4}{4K}$	$-\frac{2x^3 + x^4}{4K}$	$-\frac{2x^3 + x^4}{8K}$
Span 34 loaded.	$+\frac{x}{8K}$	$+\frac{x}{4K}$	$-\frac{x+2}{8K}$	$-\frac{x+1}{2K}$	$\frac{1}{8} - \frac{x+1}{4K}\left(1 - \frac{x+1}{2K}\right)$

Dead and Live Load Effects.—It has come to be a practice among American designers to combine dead and live loads in computing moments. This is a very unfortunate practice, because the live load effects vary greatly, whereas the dead load effects do not. To endeavor to produce total load moment factors, comprehensive enough to cover every possible ratio of dead to live load, is merely to accept a crude method, when a correct method is equally simple. In Table 3, therefore, no attempt has been made at moment-factor summation, which the writer considers incorrect in principle, inaccurate as regards results, and leading to waste in materials of construction. In addition, both dead and live loads may vary from span to span, which makes the separation of moment factors necessary. Finally, there may be variation in moments of inertia, for which case, as will be seen, separated factors are essential.

MOMENT FACTORS

In Table 3 are given the coefficients, ϕ, of the moments for any given set of dimensions and conditions of loading. To find any moment, first find the ratio, x, of the center span to the end span, then multiply the moment factors, ϕ, for spans and load conditions, by $w l^2$. Both dead and live loads for all spans must be used in this computation and the results added. Always use the outside span in the formula as all moment factors have been computed on the basis of the outside span being equal to unity.

A comparative review of the moment factors in Table 3, particularly in comparison with conventional moment factors for equal span construction, is very instructive. It is especially noteworthy that negative moments occur in the center spans whenever these are short comparatively; also, in the end spans, when they are short. Indeed, the center spans are very severely punished by the end span loads, and the span ratio has to reach 0.60 before the danger of negative moments is avoided. This point is especially significant in view of the universal custom of computing moments as positive through an erroneous use of the conventional moment factors for equal spans.

Application of Factors—Symmetrical Spans.—An example will show the practical working of the formulas and tables.

TABLE 3.—MOMENT FACTORS, ϕ, FOR MAXIMUM BENDING MOMENTS IN THREE-SPAN CONSTRUCTION.

FORMULA, $M = \phi\, w\, l^2$

(Symmetrical span arrangement; x = ratio of middle to side spans; all moment factors are computed on basis of outside spans.)

Values of x.	Values of K.	Maximum Bending Moments for:								
		Outside Spans.			Middle Spans.			Points of Support.		
		First or third span loaded.	Second span loaded.	Third or first span loaded.	First or third span loaded.	Second span loaded.	Third or first span loaded.	First or third span loaded.	Second span loaded.	Third or first span loaded.
0.20	5.72	0.078	−0.000	0.004	−0.048	0.004	−0.048	−0.105	−0.001	0.009
0.30	6.67	0.081	−0.001	0.006	−0.043	0.009	−0.043	−0.098	−0.002	0.011
0.40	7.68	0.084	−0.003	0.007	−0.039	0.015	−0.039	−0.091	−0.005	0.013
0.50	8.75	0.086	−0.005	0.007	−0.036	0.022	−0.036	−0.086	−0.009	0.014
0.60	9.88	0.088	−0.007	0.008	−0.033	0.031	−0.033	−0.081	−0.014	0.015
0.70	11.07	0.090	−0.011	0.008	−0.031	0.040	−0.031	−0.077	−0.021	0.016
0.80	12.32	0.091	−0 015	0.008	−0.028	0.051	−0.028	−0.073	−0.029	0.016
0.90	13.93	0.093	−0.019	0.008	−0.026	0.063	−0.026	−0.068	−0.038	0.016
1.00	15.00	0.094	−0.025	0.008	−0.025	0.075	−0.025	−0.067	−0.050	0.017
1.10	16.43	0.095	−0.031	0.008	−0.024	0.088	−0.024	−0.064	−0.063	0.017
1.20	17.92	0.096	−0.039	0.008	−0.022	0.103	−0.022	−0.061	−0.077	0.017
1.30	19.47	0.097	−0.047	0.008	−0.021	0.118	−0.021	−0.059	−0.093	0.017
1.40	21.08	0.098	−0.055	0.008	−0.020	0.135	−0.020	−0.057	−0.111	0.017
1.50	22.75	0.099	−0.065	0.008	−0.019	0.151	−0.019	−0.055	−0.130	0.017
1.60	24.48	0.100	−0.075	0.008	−0.018	0.170	−0.018	−0.053	−0.151	0.016
1.70	26.27	0.101	−0.087	0.008	−0.018	0.188	−0.018	−0.051	−0.173	0.016
1.80	28.12	0.101	−0.099	0.008	−0.017	0.208	−0.017	−0.050	−0.197	0.016
1.90	30.03	0.102	−0.111	0.008	−0.016	0.229	−0.016	−0.048	−0.223	0.016
2.00	32.00	0.103	−0.125	0.008	−0.016	0.250	−0.016	−0.047	−0.250	0.016
2.10	34.03	0.103	−0.140	0.008	−0.015	0.272	−0.015	−0.046	−0.279	0.015
2.20	36.12	0.104	−0.155	0.008	−0.015	0.296	−0.015	−0.044	−0.310	0.015
2.30	38.27	0.104	−0.171	0.008	−0.014	0.319	−0.014	−0.043	−0.342	0.015
2.40	40.48	0.105	−0.188	0.007	−0.014	0.344	−0.014	−0.042	−0.375	0.015
2.50	42.75	0.105	−0.205	0.007	−0.013	0.370	−0.013	−0.041	−0.411	0.015

Example 1.—A 9-ft. corridor span and two outside spans, 22 ft. 6 in. each; dead load, 67 lb. per sq. ft.; live load, 60 lb. per sq. ft for outside, and 80 lb. per sq. ft. for middle, spans; and span ratio, $x = 0.40$. From Table 3 directly:

Dead Load Bending Moments:

First and Third Spans (equal):

First span load..... $+ 0.084 \times 67 \times 22.5^2$
Second " " ..... $- 0.003 \times 67 \times 22.5^2$
Third " " ..... $+ 0.007 \times 67 \times 22.5^2$
Total effect ..... $+ 0.088 \times 67 \times 22.5^2 = + 2\,980$ ft-lb.*

Middle Span:

First span load..... $- 0.039 \times 67 \times 22.5^2$
Second " " ..... $+ 0.015 \times 67 \times 22.5^2$
Third " " ..... $- 0.039 \times 67 \times 22.5^2$
Total effect ..... $- 0.063 \times 67 \times 22.5^2 = - 2\,130$ ft-lb.

Middle Supports:

First span load..... $- 0.091 \times 67 \times 22.5^2$
Second " " ..... $- 0.005 \times 67 \times 22.5^2$
Third " " ..... $+ 0.013 \times 67 \times 22.5^2$
Total effect ..... $- 0.083 \times 67 \times 22.5^2 = - 2\,800$ ft-lb.

* The writer prefers the notation, "lb. ft." to distinguish units of moment from "ft-lb.," as units of energy.

Live Load Bending Moments:

First and Third Spans (equal):

First span load.....	+ 0.084	×	60	×	22.5^2	=	+ 2 550	
Second " "	− 0.003	×	80	×	22.5^2	=	− 120	
Third " "	+ 0.007	×	60	×	22.5^2	=	+ 210	
Maximum effect							+ 2 760	ft-lb.

Middle Span:

First span load.....	− 0.039	×	60	×	22.5^2	=	− 1 180	
Second " "	+ 0.015	×	80	×	22.5^2	=	+ 610	
Third " "	− 0.039	×	60	×	22.5^2	=	− 1 180	
Maximum effect							− 2 360	ft-lb.

Middle Supports:

First span load.....	− 0.091	×	60	×	22.5^2	=	− 2 770	
Second " "	− 0.005	×	80	×	22.5^2	=	− 200	
Third " "	+ 0.013	×	60	×	22.5^2	=	+ 395	
Maximum effect							− 2 970	ft-lb.

Total Bending Moments:

First and third spans (equal)	+ 2 980	+ 2 760	=	+ 5 740	ft-lb.
Middle span	− 2 130	− 2 360	=	− 4 490	ft-lb.
Middle supports	− 2 800	− 2 970	=	− 5 770	ft-lb.

The example is instructive in many respects. It shows how the moment factors may be summarized, when the load is equal for all spans and constant; and how they may be used separately for variable loads or loads of different magnitude. It shows that the load effects from the outside spans predominate in the middle span, so that the negative moments overbalance the positive moments. It shows how in the first span the moment from the second span load must be omitted to obtain the maximum effect and how in the middle span for the same reason its own load must be omitted. It shows, finally, that even with the outside span live loads omitted, the center span load will not be sufficient to cause a positive moment effect in the middle span as against the dead load effect from the outside spans. About most of these very important matters, conventional methods of design give no information.

Different Moments of Inertia

This leads to a consideration of three-span construction having a different moment of inertia in each span. For this purpose take Equation (3) and multiply by I_{12}; then,

$$(M_1 + 2\,M_2)\,l_{12} + (2\,M_2 + M_3)\,l_{23}\frac{I_{12}}{I_{23}} = -\frac{w_{12}\,l_{12}^3}{4} - \frac{w_{23}\,l_{23}^3}{4} \times \frac{I_{12}}{I_{23}}. \quad (34)$$

or in a more useful form, letting $q = \dfrac{I_{12}}{I_{23}}$,

$$(M_1 + 2\,M_2)\,l_{12} + (2\,M_2 + M_3)\,q\,l_{23} = -\frac{w_{12}\,l_{12}^3}{4} - q\,\frac{w_{23}\,l_{23}^3}{4}. \quad (35)$$

Expressed in words, Equation (35) means that if the moment of inertia varies from span to span, the moment factors already found may still be used, provided certain adjustments of spans and loads be introduced; that is:

(1) One span should be selected as the principal span to remain without change and with the original loads applied.

(2) The other two spans should be transformed (lengthened or shortened) in the ratio of the moments of inertia; the slenderer beams should be increased, and the heavier beams decreased, in length.

(3) The loads of the same two spans should be divided by the square of the same figure with which the spans were multiplied; or, in other words, for an increased span there should be decreased load and *vice versa*.

Application.—For purposes of demonstration and comparison take Example 1 as before, but with the moment of inertia for the middle span only a fraction of that for the outside span.

Example 2.—Same span conditions as in Example 1. Dead load, 67 lb. per sq. ft. for outside spans and 60 lb. for middle span; live loads, 60 and 80 lb., respectively; moments of inertia ratio (outside to middle spans), 4:1; middle transformed span, therefore, $4 \times 9 = 36$ ft.; span ratio, $\frac{36}{22.5} = 1.60$; modified dead load, $\frac{60}{16} = 3.75$ lb. per sq. ft.; and modified live loads, $\frac{80}{16} = 5$ lb. per sq. ft. With the original information, and with spans and loads modified, the moment factors can now be applied directly.

Dead Load Bending Moments:

First and Third Spans (equal):

First span load...	+ 0.100	×	67	× 22.5^2	=	+ 3 390	
Second " " ...	− 0.075	×	3.75	× 22.5^2	=	− 140	
Third " " ...	+ 0.008	×	67	× 22.5^2	=	+ 270	
Total effect						+ 3 520	ft-lb.

Middle Span:

First span load...	− 0.018	×	67	× 22.5^2	=	− 610	
Second " " ...	+ 0.170	×	3.75	× 22.5^2	=	+ 320	
Third " " ...	− 0.018	×	67	× 22.5^2	=	− 610	
Total effect						− 900	ft-lb.

Middle Supports:

First span load...	− 0.053	×	67	× 22.5^2	=	− 1 800	
Second " " ...	− 0.151	×	3.75	× 22.5^2	=	− 290	
Third " " ...	+ 0.016	×	67	× 22.5^2	=	+ 540	
Total effect						− 1 550	ft-lb.

Live Load Bending Moments:

First and Third Spans (equal):

First span load...	+ 0.100	×	60	× 22.5^2	=	+ 3 040	
Second " " ...	− 0.075	×	5	× 22.5^2	=	− 190	
Third " " ...	+ 0.008	×	60	× 22.5^2	=	+ 240	
Maximum effect						+ 3 280	ft-lb.

Live Load Bending Moments (Continued):

Middle Span:

First span load...	— 0.018	× 60	× 22.5²	= —	550
Second " " ...	+ 0.170	× 5	× 22.5²	= +	430
Third " " ...	— 0.018	× 60	× 22.5²	= —	550
Maximum effect				= —	1 100 ft-lb.

Middle Supports:

First span load...	— 0.053	× 60	× 22.5²	= —	1 610
Second " " ...	— 0.151	× 5	× 22.5²	= —	380
Third " " ...	+ 0.016	× 60	× 22.5²	= +	485
Maximum effect				= —	1 990 ft-lb.

Total Bending Moments:

First and third span (equal)....	+ 3 520	+ 3 280	= + 6 800
Middle span	— 910	— 1 100	= — 2 010
Middle supports	— 1 550	— 1 990	= — 3 540

Asymmetrical Span Arrangement

Thus far, only symmetrical cases have been examined. It would extend the discussion too far, and without particular value, to cover the full range of asymmetrical arrangements. The universal case may be solved without much difficulty by applying the formulas scheduled and introducing at the same time—if need be—span and load modifications as already described for variable moments of inertia.

There are numerous cases in which the span arrangement is almost symmetrical, the outside spans differing in length by a few feet only. In a moderate climate a school building, for example, may have a 10-ft. corridor in the center and 20 and 22-ft. rooms on the sides; or, in warmer climates, a 20-ft. room in the center with 12 and 14-ft. verandahs on the sides.

A survey of the moment factors in Table 3, particularly for the supplementary load effects from adjacent spans, will make it plain that such minor variations from true symmetry will not cancel the usefulness of the tabular figures. Each end span, with its inner support, may be computed quite accurately as if the opposite side were the same and, then, the middle span, assuming the end spans to be equal to an average of the two.

Assume, for instance, spans of 20 ft. 0 in., 9 ft. 0 in., and 22 ft. 0 in. The spans ratios are then 0.45 and 0.41, respectively. For these differences the variations in the load effects are visible to a small extent in the third decimal only. A simple example using a larger asymmetry would readily show further approximation.

Conventional Errors

In usual designing practice the solution by the graphical method or the three-moment theorem is not very common; instead, conventional moment factors are ordinarily applied. These factors were never intended to apply except for equal spans. How dangerously incorrect the use of these factors

is, may be shown by continuing the foregoing examples in accordance with the conventional method as follows:

$$\text{Outside spans} \quad \frac{127 \times 22.5^2}{10} = +\ 6\ 430 \text{ ft-lb.}$$

$$\text{Middle spans} \quad \frac{140 \times 9^2}{12} = +\ 940 \text{ ft-lb.}$$

$$\text{Middle supports} \quad \frac{-133 \times 15.75^2}{12} = -\ 2\ 750 \text{ ft-lb.}$$

A comparison of these results with those previously obtained is enlightening. A summary of the three methods is shown in Table 4.

TABLE 4.—COMPARISON OF MOMENTS, IN FOOT-POUNDS, BY VARIOUS METHODS.

Location.	By Three-Moment Theorem:		By conventional moment factors.
	Equal values of *I*.	Unequal values of *I*.	
Outside spans	+5 740	+6 800	+6 430
Middle spans	−4 490	−2 010	+ 940
Middle supports	−5 770	−3 540	−2 750

PRACTICAL CONSEQUENCES

Obviously, for the example chosen—and it represents a very common case—the moments for both the middle span and its support are most erroneously calculated. This is true not only with the rough conventional moment factors, but also when the three-moment theorem is applied without allowance for the change in cross-section.

The middle span moment by the conventional method is positive, whereas it should be decidedly negative and increase by a parabolic curve until the still larger support moment is reached. By the conventional method the support and outside span moments are also incorrect, but in this example not dangerously so. In regard to the use of the three-moment theorem it should be noted that modification for the change of section is most necessary, as the differences in results are very large—for the middle span and support moments, 55% and 49%, respectively. In this case, it is true the error is on the safe side, but with other span proportions it would be the reverse.

At any rate a question of economy or strength has been raised: Who would advocate using carelessly large quantities of materials when they are not needed, or leaving them out when they are required for strength? The question might also be asked: What chance, for instance, has a slender slab of concrete, proportioned for a moment of 940 ft-lb., to resist a negative moment of 3 540 ft-lb.?

Careless Designs.—The ignorance or lack of care in this matter would have brought disastrous results in reinforced concrete construction, had it not been for compensating features. When the two outer spans are long and the middle spans short, or *vice versa*—the cases where conventional methods are

most in error—the heavier spans with their supports generally act as independent units and as such are strong enough to stand of themselves. The slenderer spans also act independently, and are able somehow to hang on to the supports even in spite of numerous cracks, which are not as large as they might be (on account of generous load compensations) and are efficiently concealed by laitance, dust, floor coverings, partitions, base-boards, and the like.

Revision of Current Practice Essential.—This general subject has been taboo both to university and to municipal authorities. Consulting engineers likewise seem to be unable to spare the time to design correctly. The large reinforcement bar distributors, "pioneers and consultants in reinforced concrete," are possibly the worst offenders. Their various "tin pan systems", most excellent for long-span construction, are lagging behind in details suitable for a correct computation of continuity.

Perhaps even the Joint Committee on Concrete and Reinforced Concrete is somewhat to blame. It would have been consistent and quite in order for this Committee to omit all reference to moment factors, which are not truly subject to regulation by anything akin to a code; but to write several pages about moments for equal spans and then to state* merely that "continuous beams with unequal spans * * * shall be designed for actual moments", is surely inconsistent and only invites others to "escape from an unpleasant reality".

One effect on bureaucratic minds has been to make the conventional moment factors universal for almost all spans and conditions. Indeed a competent designer is sometimes forced to do what he knows to be wrong, because some one in authority has spoken with the appearance of great understanding. Possibly, and conceivably, this paper may bring some of these hazy or incorrect ideas to a proper focus.

Bibliography

The following bibliography on three-span construction has been prepared by the writer. Each reference was selected for one of three reasons: (*a*) The information is fundamental and classic; (*b*) the accepted theories have been expressed; and (*c*) the theories have been carried to practical results, with working formulas and tables.

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DISCUSSION

FRANK S. BAILEY,* ASSOC. M. AM. SOC. C. E. (by letter).—Continuous beams have been studied so extensively in the past that the writer wondered when seeing the title of the author's paper what new treatment of the subject was possible. A reading of the paper, however, is sufficient to convince any designer of its value in eliminating the necessity for laborious computations in many cases.

Table 3, giving moment factors for maximum bending moments in three-span construction, should be a very real labor saver. A table giving the moment factors for three spans with fixed ends, would also be useful.

In a short paper entitled "Diagrams for Continuous Beams of Unequal Spans,"† the writer presented several curves of shears and moments for two and three-span beams, with supported and also with fixed ends, and generally carrying a uniform load over all spans. It is interesting to check the results in Table 3, with those found by the writer. The moment factors check with each other almost exactly for the case where all three spans are loaded. It should be noted that Table 3 gives factors for maximum bending moments but, while it gives a negative moment of $0.05\ w\ l^2$ as the maximum for the middle span in the case where the value of x is 0.50, this is really the minimum value of the moment, and occurs at the center of the span, as shown on Fig. 4 of the writer's paper previously mentioned, a negative moment extends entirely across the middle span and is a maximum adjacent to the supports.

In comparing Fig. 3 of the writer's paper referred to, with the author's Table 3, it was found that the moments at the center of the middle span and at the two supports check very closely, but as a matter of fact, the maximum moment in the middle span does not occur at the center, but adjacent to the supports, where it is negative. This fact, of course, would be obvious by using Table 3 and plotting a diagram showing the moments at the supports as well as in the spans.

The maximum positive moment in the end span as shown on the writer's Fig. 3 is equal to $+\ 0.024\ w\ l^2$, whereas Table 3 shows a maximum moment of $-\ 0.014\ w\ l^2$ in the end span, for all spans loaded. Here, again, the location of the maximum moment is really adjacent to the interior support.

The writer found a small positive moment in the end span while Table 3 gave a small negative moment as the maximum. On further reference to the paper the writer read that, "it is necessary to introduce an approximation, by assuming that the secondary effects at the mid-spans will give results close enough to be added to the two principal moments." It is likely, therefore, that this approximation may account for the rather unimportant discrepancy.

It is to be hoped that the author's criticisms of existing building codes and other standard rules for moment factors, will have beneficial effects. Sponsors of the various codes undoubtedly wish them to be as trustworthy as possible, and defects when found will be corrected sooner or later.

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† *Journal*, Boston Soc. of Civ. Engrs., Vol. 4, No. 8, October, 1917.

Despite the best of building codes, accidents will happen if building inspectors and other officials and contractors are not extremely vigilant and perhaps sometimes even if they are. For several years the City of Boston has had a building code which will probably stand comparison with any other, and yet, because of some unforeseen weakness, a building in which a dance was being held a few years ago, suddenly collapsed, with shockingly tragic results.

. CLYDE T. MORRIS,* M. AM. SOC. C. E. (by letter).—It is with gratification that one sees the tendency of designing engineers in recent years to calculate stresses with greater precision. This tendency has been especially marked in reinforced concrete design, probably due to the fact that its basic theory is comparatively new and has been developed in connection with tests in university laboratories, by research workers skilled in this type of investigation. The uncertainties of the physical properties of concrete, namely, modulus of elasticity, time yield, and ultimate strength, would not seem to call for as careful theoretical computation of stresses as might be advantageous in the older material—steel—which has well known and constant physical characteristics. In spite of this, steel structures are still designed on assumptions of simple beams, girders, and trusses, connected by hinged joints. There is no doubt in the writer's mind that the steel frame of a modern, well-designed office building is just as much a monolith, as the building with a reinforced concrete frame. In fact, the effect of time yield of concrete upon the negative dead load moments at the joints of a reinforced concrete frame must be to relieve them to a considerable extent and to increase the positive moments at the mid-spans.

Therefore, it would seem that careful consideration of continuity in a steel frame with well-designed joints is just as important as in a reinforced concrete frame. Usually, the joints of steel frames must be designed to resist wind moments, and if properly designed will also care for the joint moments caused by dead and live loads.

It has been the writer's practice, in designing steel frame buildings, to make use of the same diagrams and methods developed for calculating the moments at the joints of rigid frames, which are published in the various texts on reinforced concrete construction.

Mr. Oesterblom illustrates well the fallacy of designing by rule-of-thumb methods. The writer would go even further and take into account the restraining effect of the supports. It is seldom that a continuous beam is supported on joints which are free to turn. It is usually rigidly connected to the columns, and the bending moments in both must be calculated and the design proportioned accordingly.

In a bent, with equal or nearly equal story heights, the slope deflection equations for any one floor may be written independently of the other floors, by assuming that the angles of rotation of the joints immediately above and below are equal to those at the floor under consideration. This is equivalent to assuming a hinged-end condition in all the columns at the mid-story heights. In a symmetrical three-span bent, such as that described by the

* Prof. of Structural Eng., Ohio State Univ., Columbus, Ohio.

author, this results in two equations and two unknown slopes when the loads are symmetrical, as they are for maximum span moments and for dead load.

A. Lincoln Hyde,* M. Am. Soc. C. E. (by letter).—The author is to be commended for his efforts to simplify, for practicing structural engineers, the frequently occurring problem of continuous beams over three spans, symmetrical with respect to end spans, but with the middle span varying from 0.2 to 2.5 times the end span. The limits of his values of x seem to be well chosen. For office buildings, hotels, hospitals, etc., they will fall well below 1.0, but for highway bridges in locations where it is desired to provide wide middle channels, the values of x may approach 2.5.

The writer feels that some designers will be saved considerable work by the use of the author's table of moment factors (Table 3), but designers of reinforced concrete are vitally interested in maximum shears and in the location of inflection points as well as in maximum bending moments.

For his own use the writer has prepared tables of reaction coefficients for beams continuous over three spans, the end spans being equal, and the middle span varying from 0.2 to 2.5 times the end span. With these tables the designer with little labor, may compute maximum bending moments and maximum shears, and may also locate inflection points. Tables 5 and 6 are herewith submitted with the hope that they may supplement Table 3, and prove of value to other designers. They were developed from the well-known Merriman formulas for a symmetrical span arrangement; x is the ratio of the middle spans, and all coefficients are computed on the basis of outside spans.

To find any reaction, first find the ratio, x, of the middle span to the end span, and then multiply the reaction coefficient, θ, by $w\,l$. The value, w, must be the dead load or the live load per linear foot as the case may be, and the value, l, must be the length of the end span, in feet. The dead and live load effects must be added for total effect.

Application of Reaction Coefficients.—Using the data of the author's Example 1, and dealing with Span 12: For full uniform dead load, and uniform live load over both end spans,

$$R_1 = 0.417 \times 67 \times 22.5 + 0.4218 \times 60 \times 22.5 = 630 + 570 = 1\,200 \text{ lb.}$$

The section of zero shear and maximum bending moment is distant to the right of Support 1,

$$\frac{630 + 570}{67 + 60} = 9.45 \text{ ft.}$$

The maximum bending moment is,

$$M_{12} = 1\,200 \times 9.45 - 127\,\frac{(9.45)^2}{2} = 5\,670 \text{ ft-lb.}$$

which checks the author within 1 per cent.

The inflection point is distant to the left of Support 2,

$$22.5 - 2 \times 9.45 = 3.60 \text{ ft.}$$

The shear at Support 1 is 1 200 lb., and at the left of Support 2, it is,

$$1\,200 - 22.5 \times 127 = -1\,660 \text{ lb.}$$

* Prof. of Bridge Eng., Univ. of Missouri, Columbia, Mo.

TABLE 5.—Reaction Coefficients in the Formula, $R = \theta wl$.

Values of Coefficient, θ.

Full Uniform Load.			Uniform Load, Middle Span.		
Values of x.	R_1 and R_4.	R_2 and R_3.	R_1 and R_4.	R_2 and R_3.	Values of x.
0.2	0.4031	0.6969	−0.0007	0.1007	0.2
0.3	0.4115	0.7385	−0.0022	0.1522	0.3
0.4	0.4169	0.7831	−0.0050	0.2050	0.4
0.5	0.4196	0.8304	−0.0090	0.2590	0.5
0.6	0.4200	0.8800	−0.0142	0.3142	0.6
0.7	0.4181	0.9319	−0.0209	0.3709	0.7
0.8	0.4140	0.9860	−0.0291	0.4291	0.8
0.9	0.4080	1.0420	−0.0388	0.4888	0.9
1.0	0.4000	1.1000	−0.0500	0.5500	1.0
1.1	0.3900	1.1600	−0.0628	0.6128	1.1
1.2	0.3782	1.2218	−0.0771	0.6771	1.2
1.3	0.3645	1.2855	−0.0931	0.7431	1.3
1.4	0.3490	1.3510	−0.1106	0.8106	1.4
1.5	0.3317	1.4183	−0.1298	0.8798	1.5
1.6	0.3126	1.4874	−0.1506	0.9506	1.6
1.7	0.2918	1.5582	−0.1730	1.0230	1.7
1.8	0.2692	1.6308	−0.1970	1.0970	1.8
1.9	0.2448	1.7052	−0.2227	1.1727	1.9
2.0	0.2188	1.7812	−0.2500	1.2500	2.0
2.1	0.1909	1.8591	−0.2789	1.3289	2.1
2.2	0.1614	1.9386	−0.3095	1.4095	2.2
2.3	0.1301	2.0199	−0.3418	1.4918	2.3
2.4	0.0972	2.1028	−0.3756	1.5756	2.4
2.5	0.0625	2.1875	−0.4111	1.6611	2.5

TABLE 6.—Reaction Factors in the Formula, $R = \theta wl$.

Values of Coefficient, θ.

Uniform Load, Left-End Span.					Uniform Load, Both End Spans.		
Values of x.	R_1.	R_2.	R_3.	R_4.	R_1 and R_4.	R_2 and R_3.	Values of x.
0.2	0.3951	1.1731	−0.5769	0.0087	0.4038	0.5962	0.2
0.3	0.4025	0.9598	−0.3735	0.0112	0.4137	0.5863	0.3
0.4	0.4089	0.8516	−0.2735	0.0130	0.4219	0.5781	0.4
0.5	0.4143	0.7857	−0.2143	0.0143	0.4286	0.5714	0.5
0.6	0.4190	0.7412	−0.1754	0.0152	0.4342	0.5658	0.6
0.7	0.4232	0.7091	−0.1481	0.0158	0.4390	0.5610	0.7
0.8	0.4269	0.6847	−0.1278	0.0162	0.4431	0.5569	0.8
0.9	0.4303	0.6655	−0.1123	0.0165	0.4468	0.5532	0.9
1.0	0.4333	0.6500	−0.1000	0.0167	0.4500	0.5500	1.0
1.1	0.4361	0.6372	−0.0900	0.0167	0.4528	0.5472	1.1
1.2	0.4386	0.6265	−0.0818	0.0167	0.4553	0.5447	1.2
1.3	0.4409	0.6174	−0.0750	0.0167	0.4576	0.5424	1.3
1.4	0.4430	0.6095	−0.0691	0.0166	0.4596	0.5404	1.4
1.5	0.4450	0.6026	−0.0641	0.0165	0.4615	0.5385	1.5
1.6	0.4469	0.5965	−0.0597	0.0163	0.4632	0.5368	1.6
1.7	0.4486	0.5911	−0.0559	0.0162	0.4648	0.5352	1.7
1.8	0.4502	0.5863	−0.0525	0.0160	0.4662	0.5338	1.8
1.9	0.4517	0.5820	−0.0495	0.0158	0.4675	0.5325	1.9
2.0	0.4531	0.5781	−0.0469	0.0157	0.4688	0.5312	2.0
2.1	0.4544	0.5747	−0.0445	0.0154	0.4698	0.5302	2.1
2.2	0.4557	0.5714	−0.0423	0.0152	0.4709	0.5291	2.2
2.3	0.4569	0.5684	−0.0403	0.0150	0.4719	0.5281	2.3
2.4	0.4580	0.5657	−0.0385	0.0148	0.4728	0.5272	2.4
2.5	0.4590	0.5632	−0.0368	0.0146	0.4736	0.5264	2.5

For Span 23 the loading for maximum bending moment is the same as before. The reaction at Support 2 is,

$$R_2 = 0.783 \times 67 \times 22.5 + 0.5871 \times 60 \times 22.5 = 1\,960 \text{ lb.}$$

and,

$$M_{23} = 1\,200 \times 27 + 1\,960 \times 4.5 - 127 \times 22.5 \times 15.75 - 67 \times 4.5 \times 2.25 = -4\,470 \text{ ft-lb.}$$

which checks the author closely.

The bending moment being negative over the entire middle span, there will be no inflection point between Supports 2 and 3. The shear at the right of Support 2 is,

$$1\,200 + 1\,960 - 2\,860 = 300 \text{ lb.}$$

The maximum bending moment at Support 2 occurs under the full uniform dead load, and uniform live loads over Spans 12 and 23. The reaction at Support 1 for this loading is,

$$R_1' = 630 + 0.4089 \times 60 \times 22.5 - 0.0050 \times 80 \times 22.5 = 1\,170 \text{ lb.}$$

and,

$$M_2 = 1\,170 \times 22.5 - 127 \times 22.5 \times 11.25 = -5\,820 \text{ ft-lb.}$$

which again checks the author closely.

HAROLD E. WESSMAN,* ASSOC. M. AM. SOC. C. E. (by letter).—This and other recent papers† relating largely to moments in continuous spans are focusing the attention of structural engineers more and more on "statically indeterminate" structures. Scientific research has been stimulated and means are being sought to simplify methods of analysis, so that the designer in continuous structures may determine his unknown forces without going through long, involved, tedious, and costly computations. There has been a tendency in some circles to allow for the unavoidable presence of continuity by large factors of safety or arbitrarily selected coefficients, which have been established by well-meaning authorities for use in specific cases only. The writer believes that within the next few years there will be radical developments in this field of analysis, which will not only check the tendency to shroud the so-called "indeterminate" structure in a cloak of mystery, but will present it in such a light that design practice will know it better and use it more intelligently. Some engineers apparently believe that "statically indeterminate" structures are "absolutely indeterminate" structures and cannot be analyzed by any means.

Mr. Oesterblom, however, makes a rather sweeping indictment of universities, municipal authorities, and consulting engineers on what he states is their current practice in the teaching of design or the actual design of continuous structures. No doubt there are cases warranting his indictment, but general conclusions are not justified. The writer has been connected with all

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† "Moments in Restrained and Continuous Beams by the Method of Conjugate Points", by L. H. Nishkian and D. B. Steinman, Members, Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 90 (1927), p. 1; and "Distribution of Reinforcing Steel in Concrete Beams and Slabs," by Boyd S. Myers, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 91 (1927), p. 174.

three groups mentioned by the author and knows that representatives of each* are facing the facts of design for continuous structures as they exist and are not adhering blindly to those conventional rules found particularly in reinforced concrete design codes and which apply only in a limited number of cases.

The author takes the familiar three-moment equation in a form similar to that presented by Clapeyron and from it derives expressions for the moments at the critical points in a three-span system due to loading one span at a time with a unit uniform load per foot. The two outside spans are alike and of length unity and the center span is of variable length, x. The critical points are at the supports and at or near the center of each span. For the system outlined, the three-moment equation is not particularly burdensome and lends itself readily to the computation of a table such as the author makes (Table 3); but when the systems are decidedly unsymmetrical, when they have three or more spans, and involve variable moments of inertia, this method becomes very complex and tedious. Moreover, when the beams or girders are framed into restraining columns or walls at the supports, it cannot ordinarily be used at all. The writer is familiar with several methods for computing moments and shears in continuous systems. The one which he prefers, is extremely flexible, may be readily applied regardless of the number of spans, varying moment of inertia, or restraint at supports, and gives accurate results with remarkable rapidity. This method has been used in checking the author's Table 3 and in obtaining results which follow.

The author loads one span at a time and computes "moment factors" for the point of maximum moment in the loaded span, for the mid-points of the unloaded spans, and for the supports. In the middle span, the point of maximum positive moment is the center of the span; but in each outside span when loaded, it is approximately at 0.41 from the support instead of at the center. For the effect in the same span due to loads on the other spans, the author computes the moment ordinate, not at the same point where his maximum positive moment occurred, but at the center of the span. For the combined moment due to loading all the spans, he consequently adds moment ordinates which do not occur at the same point. An inspection of Fig. 2 will make this clearer. This procedure is somewhat inconsistent, but the author justifies it by stating that, "on all the spans the lines representing the two unloaded effects are compensating, and the resultant of the two is almost horizontal." That is only true for the outside spans when $x < 0.60$. (See Table 3.) When $x > 0.6$, the secondary $-M$ in the first span, due to the second span load, becomes very much greater than the secondary $+M$ in the first span due to the third span load.

The total combined moment is not exact (as the author states), but it is close enough for practical purposes, being 1% in error when the ratio of center span to end span is 0.5 and about 4% in error when all the spans are equal in length. If all ordinates had been determined at the mid-points, the error would vary within nearly the same range.

* "Structural Design of Bridges in Grant Park," by C. R. Hoyt, Assoc. M. Am. Soc. C. E., *Journal*, Western Soc. of Engrs., October, 1925; and, "The South Park Boulevard Viaduct," by T. L. Condron, M. Am. Soc. C. E., *loc. cit.*, July, 1923.

Moreover, the author's coefficients are based on the assumption of knife-edge supports. Even in slabs and beams not framed into columns or deep girders, the actual conditions give area support rather than line support. In addition, they develop some restraint, the effect of which is to reduce the $+ M$ in the center of the span and in some cases the $- M$ at the support.

Those responsible for the codes of the Joint Committee on Specifications for Concrete and Reinforced Concrete, both old and new, and the American Concrete Institute Code may have had this effect in mind when they specified the clear span as the effective length to be used in determining moments. Definite information on this point apparently is difficult to obtain.

In view of the foregoing, the writer believes that the author could have saved himself some work by computing the span moments at the mid-points only and at the supports. The results are accurate enough for those practical purposes for which the author's moment factors may be used, namely, the design of three-span slabs, and beams symmetrical, or approximately so, about the center span and with little or no restraint at the supports.

The author illustrates the use of his table of moment factors (Table 3) by application to a typical design problem, a three-span system with a 9-ft. corridor span and two outside spans, 22 ft. 6 in. each. After finding total maximum positive and negative moments for dead load over all spans and moving uniform live load, he compares them with the moments obtained by using the conventional coefficients, $\frac{1}{10}$ and $\frac{1}{12}$, which were never intended to apply to anything but equal spans. Naturally, there is a large discrepancy in the two results. (See Table 4.) The author points to this as a very common case in which moments are most erroneously calculated. The writer feels that the case is the exception rather than the rule, for he doubts very much whether the average designer would try to apply the conventional coefficients for equal spans to systems similar to that of the author, in which the center span is so much shorter than the end spans. The picture of the physical action of the short center span would be apparent enough to most designers to warn them of high negative moment at the supports and the possibility of negative moment over the entire center span.

The author calls attention to the value of the moment factors in showing the effect on any critical point due to loading in any one span and how certain span loads must be omitted to obtain maximum effects. The writer would like to call attention to the value of qualitative influence lines for determining the placing of live loads to cause greatest effect at the points to be considered. For those to whom the concept of the influence line as the shape of the deformed structure when the particular effect is acting at the point under investigation is familiar, the sketching of the shape of the influence line is only a moment's task. The positive and negative areas under the influence line then indicate the position of live loads to give maximum positive and negative effects.

Fig. 3 shows influence lines for moments at various governing points in a three-span system. Loading conditions for maximum positive and negative

effects are also indicated. For the three-span system, positions of loading are apparent to some designers without drawing influence lines, but where there are more than three spans, and especially where there is multiple-bent or multi-story framing, qualitative sketching of influence lines is of great value. In most cases, only the sketch is needed; but, occasionally, where maximum moments or shears due to a small number of heavy moving concentrated loads is wanted, it may be advisable to determine the actual influence ordinates, in other words, to find the quantitative influence line. It is a matter of incidental interest to note that the author's moment factors are measures to some scale of the area beneath the influence lines, the scale depending on the ratio of span lengths.

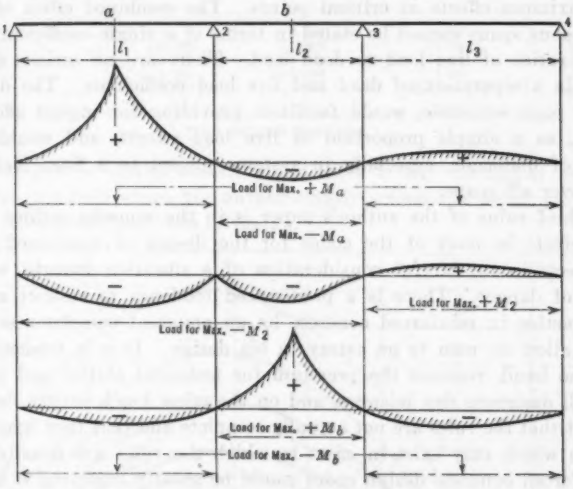

FIG. 3.—INFLUENCE LINES FOR MOMENTS AT CRITICAL POINTS.

The author is to be commended for calling attention to the possible existence of negative moments over the entire center span, but he sets his point of danger too low. He states,

"Indeed, the center spans are very severely punished by the end span loads, and the span ratio has to reach 0.60 before the danger of negative moments is avoided."

The author overlooks the effect of varying the ratio of $\frac{\text{live load}}{\text{dead load}}$. This does not have to be large or unusual to cause negative moment over the entire center span, even when the center span is greater in length than the outside spans. Moreover, as the span ratio becomes greater than 1.0, negative moment may occur along the entire length of the outside span, necessitating tension steel all the way across the top as well as in the bottom of slabs, beams, and girders. This is a fact which is often overlooked in design, particularly in equal span systems, because they are usually designed in

accordance with codes, in which the moment coefficients specified completely ignore the great possibility of the existence of negative moment at the center of the spans. Conditions very often arise, that require tension reinforcing in the top across the entire length of the system.

The author is also to be commended on his plea for the separation of dead load from live load in computing moments. In steel design, they are usually separated, but in reinforced concrete design, it seems to be quite general practice to combine the two. The writer believes this is due largely to the prevailing codes of design for reinforced concrete which specify moment coefficients to be used for combined dead and live load.

Dead load is present over all spans; live load is placed only where it will cause maximum effects at critical points. The combined effect of the two in continuous spans cannot be stated in terms of a single coefficient that will cover all ratios of live load to dead load. There are no serious difficulties involved in a separation of dead and live load coefficients. The distinction would be more scientific, would facilitate providing for impact effects when necessary, as a simple proportion of live load effects, and would promote economy of materials, especially in systems subject to a fixed loading that extends over all spans.

The chief value of the author's paper is in the remarks calling attention to the defects in most of the codes for the design of reinforced concrete. Such defects invite careful consideration of a situation fraught with some elements of danger. There is a pronounced tendency to restrict and guide design practice in reinforced concrete by an apparent completeness of rules that will allow no man to go astray in his design. It is a tendency which, on the one hand, removes the premium for technical ability and judgment, making all designers day laborers, and on the other hand, invites danger due to the fact that the rules are not actually complete and that they ignore actual conditions which may exist in cases to which the rules are usually applied. The reinforced concrete design codes would be greatly improved if they were patterned more closely after existing specifications for steel design which do not attempt to "corral" structural analysis.

ROBERT M. WILDER,* JUN. AM. SOC. C. E. (by letter).—The writer thoroughly agrees with the author as to the danger of neglecting such important factors as variability in span lengths, loads, and sections. The author has very ingenuously evolved a table (Table 3) to facilitate the calculation of moments in continuous beams with due regard to these factors.

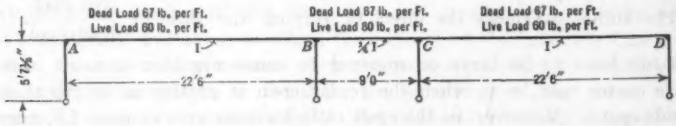

FIG. 4.

One factor which influences the moments in beams just as certainly, has been entirely neglected; that is, the restraint offered at the support to angular

* Asst. Supt., Seaboard & Southern Const. Co., Jacksonville, Fla.

deflection of the beam at that point. This restraint may be due to the supporting columns in the case of beams, or to the supporting beams in the case of slabs.

TABLE 7.—MAXIMUM MOMENTS IN FIG. 4, DUE TO VARIOUS DEGREES OF COLUMN RESTRAINT, IN FOOT-POUNDS.

Maximum moment at Point.	COLUMN RESTRAINT OR MOMENT OF INERTIA OF COLUMNS DIVIDED BY MOMENT OF INERTIA OF END BEAMS.			
	0*	$\frac{1}{8}$	$\frac{1}{2}$	∞
A (toward *B*)	0	2 080	−3 870	−5 360
Center *A B*	±6 800	±5 200	±3 880	±2 680
B (toward *A*)	−3 540	−4 200	−4 800	−5 360
B (" *C*)	−3 540	−2 680	−1 830	− 990
Center *B C*	−2 010	−1 200	−460, or +220	± 500

* No column restraint.

This restraint is seldom absent where there is any continuity, especially in buildings, the field which the author rightly stressed as being particularly subject to "handbook" methods of design.

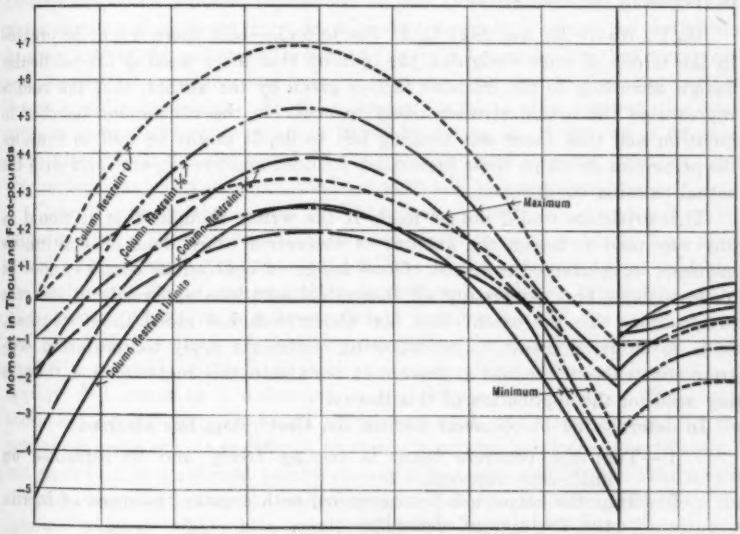

FIG. 5.—EFFECT OF COLUMN RESTRAINT ON THREE-SPAN BEAMS.

To determine the actual effects of column restraint, the writer used the same illustration presented by the author, added various conditions of column restraint, and calculated the maximum moments. This case is shown in Fig. 4. It might be noted that within the limits of practical computation the results would have been the same with columns above as well as below, their

moments of inertia being one-half those used in the case of columns below only. The results of the calculations are given in Table 7.

The results are shown graphically in Fig. 5. The upper curve in each case is for loadings producing maximum moments; the lower, minimum moments. Only one-half of the three spans is shown, as the other half would be symmetrical.

The cases of restraint used are quite common in any building and, in most instances, the values would be between the nominal value, ½, and infinity. It can thus be seen that neglecting the effect of column restraint is quite as dangerous as neglecting the other factors.

In the case of concrete construction still another factor presents itself, which was not noted in this paper, namely, the effect of moments of inertia varying from center to support in the same beam.*

The arbitrary moment factors used in so many specifications and building codes are admittedly intended only for certain general conditions of spans, loads, and sections. Where the actual conditions depart from these general conditions to such an extent that the code cannot be modified by ordinary judgment, the designer should make a careful analysis of the structure, taking into consideration not only the factors of continuity, span, load, and section, but also such important factors as restraint at supports and varying moments of inertia in the same span.

W. F. Way,† M. Am. Soc. C. E. (by letter).—Lest there might be created in the minds of some designers, the opinion that after making his particular design according to the moment factors given by the author, that the results represented the actual stresses likely to occur in the continuous member in question and that there was nothing left to do, it might be well to consider the principles on which these factors are founded and to compare them with the actual existing conditions.

This criticism would not be made if the writer thought that it would in any way tend to lessen the amount of theoretical work done on continuous members in reinforced concrete. Good design in continuous beams cannot be made without first exhausting all theoretical solutions applicable to the case in question. To the extent that Mr. Oesterblom has aided these solutions, he is to be given credit. The following comments apply to reinforced concrete construction because at present it is only in this material that there is any need for the application of this theory.

In determining his moment factors Mr. Oesterblom has assumed:

1.—That the concrete beam is resting freely and unrestrained on knife-edge supports.
2.—That the beams are homogeneous, with constant moment of inertia and modulus of elasticity.
3.—That the stresses in the beams are unaffected by shrinkage.
4.—That the temperature stresses are negligible.

Consider these assumptions in order to see how nearly they parallel actual concrete construction.

* The effect of this factor was shown by Mr. R. E. Spaulding in an able article in *Engineering News*, January 13, 1910.

† Constr. Engr. for Henry & McFee, Seattle, Wash.

Instead of the beam resting freely on a knife-edge support, more often it is resting on a building column and frequently on haunches. In addition, there is often a column above the beam continuous with the support. In this case the author's moment factors are certainly altered, the amount depending on story height, spans, and moments of inertia of columns and beams. In a few instances these factors may be small enough to be neglected, but not in many; more often they will alter the factors considerably. Building codes and most textbooks alike neglect this case, the only one suggesting a method for the amount of negative moment in a continuous beam over a wide support, that has come to the writer's attention, is given in "Beton Kalender."* In important and repeat work this method should be only one of several used in arriving at a fair average for the proper figure for design.

Referring to the second assumption, it is true that a reinforced continuous concrete beam is not composed of a homogeneous material, nor has it a constant moment of inertia or modulus of elasticity. In most instances, the difference between this assumption and actual conditions is probably so small that it may be neglected. However, the facts should be realized as it is not improbable that they may materially affect the moment factors in special cases.

It is well known that concrete shrinks considerably during the early period of hardening. This shrinkage induces considerable compression in the reinforcing steel—the amount depending on the steel percentage, cement used, water content of concrete, etc. In a precast concrete project† the writer set gauge points in the main reinforcement for forty beams before they were cast, and observed the compression induced into the steel due to shrinkage of the concrete while hardening. In these beams, tested in this manner, the compression amounted to an average of 12 000 lb. per sq. in. at the end of 60 days. Even after placing these beams in the structure and pouring the slab, thus placing full dead load on them, the bars were still in compression.

There is even another aspect to this property of reinforced concrete which probably affects the stresses in continuous beams more than in the case mentioned. For example, assume a tall building of the apartment type, in which the continuous beams are supported by two wall and two interior columns. The usual construction is a large column forming part of the exterior wall and very lightly reinforced, while to save space the interior column is made as small as possible with a high steel percentage. This difference in steel percentage will result in a difference in the shrinkage coefficient. Since this coefficient depends on the brand of cement used, the water content in the poured concrete, etc., it is difficult to assign a general value for this difference, but according to published results it might be taken as 0.0003. This means that for every 10-ft. story the wall column will shorten 0.036 in. more than the interior columns. This is a small difference but in a 7-story building it amounts to a total of ¼ in. Even this is not realized in buildings, but it must be acknowledged that some force is required to reduce this difference and that this same force is altering the stresses in the continuous beams tying the columns together, which added to the stresses induced into the continuous

* 1923 Edition, p. 305.

† *Engineering News-Record*, March 13, 1924.

beams, due to difference in column shrinkage, would be of sufficient magnitude to alter the theoretical moment factors considerably.

As for temperature stresses in reinforced concrete building construction, the effect is similar to the case of shrinkage stresses. Take the apartment house example again and assume the exterior walls to be of concrete, stuccoed. During the winter season the difference between the temperature of the interior and exterior columns may easily be as much as 60° Fahr., which would mean that for every 10-ft. story there would be a difference of 0.043 in. in the expansion between the interior and exterior columns. During the summer seasons this condition might easily be reversed. The changing elevations of the supports of the continuous beams will most certainly alter the moment factor, and the temperature stresses added to the shrinkage stresses will most likely be of measurable magnitude.

In the closing paragraphs of the paper severe criticism is made of the report of the Joint Committee on Specifications for Concrete and Reinforced Concrete, principally because no definite regulation was made for "continuous beams with unequal spans". A similar criticism might well be made of the author because he has offered no solution whatever for continuous beams with fixed ends. In general construction it is seldom that continuous beams with free ends occur. They are usually at least partly fixed if not entirely, and it is quite as important to have a solution for one case as for the other.

Since it is impossible to make an accurate mathematical design for a reinforced concrete beam, empirical factors must be used for building codes. Committees writing these codes have realized this and, as a member of the Building Code Committee of Seattle, Wash., the writer felt no chagrin at Mr. Oesterblom's severe criticism.

Reinforced concrete buildings have been built and are still being constructed that have and will give satisfaction. It is not the writer's intention in the least to discredit properly designed and constructed reinforced concrete structures. It is the intention of this discussion to show that the assumptions on which Mr. Oesterblom based his calculations for his moment factors are far from being realized in actual construction in reinforced concrete. It is well that these limitations should be clearly understood by the designer, and, while it may be impossible to solve their numerical influence exactly, still this influence can at least be sensed by the designer and some provision made in the finished structure to carry these stresses.

H. M. HADLEY,* ASSOC. M. AM. SOC. C. E. (by letter).—The author's discussion of and comments on the practical present-day methods of designing continuous beams very happily describes and appraises them. They are arbitrary; even capricious. The only thing to be said in their favor is that they are simple, and that things are accomplished by them. The designer looks at his spans and his loads; considers for a moment; decides that $w\,l$ over something is about right; and forthwith proceeds with his computations. An experienced man will form judgments that may quite closely approximate the theoretical stress distribution; the judgments of the inexperienced man

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will not be so close. In either event some definite result will be obtained and some decision reached that will find expression in designated sizes of beams, specified quantity of reinforcement, or similar directions. Such designs, when built and submitted to the actual test of gravity, will in most cases appear to be satisfactory. Nothing alarming happens; a few cracks may develop; nothing more. The construction stands up and carries applied loads without pronounced signs of distress, and, in the busy rush of affairs, with these results the designers are, perforce, content.

Probably no one is wholly pleased with such methods of design, but in the absence of reasonably simple and economical means of obtaining a correct solution of the problem, the arbitrary and expeditious ways of designing continuous beams will doubtless continue to find favor.

Mr. Oesterblom's selection of the continuous beam of three spans as the one of outstanding importance is well taken. It is of more frequent occurrence than any other, and in multi-storied, typical construction that is repeated over and over again in the same building, this specific problem, if any, is deserving of a correct solution, because the economies that may be effected thereby are multiplied manyfold. Similarly, such buildings offer opportunities for repeated mistakes.

In giving separate consideration to dead and live loads and in treating the spans individually, determining the separate moment effects, and combining them for maximum conditions, Mr. Oesterblom's treatment is most excellent and logical. The effect of loads on adjacent spans is made clear and plain by this method, and one can see the influence of the several parts in the combined result.

The statement that moment factors are not properly a subject for regulation by anything akin to a code has an engaging interest. King Canute and the rising tide come to mind. Will the day come when $\frac{w\,l^2}{12}$ will no longer be established and ordained in American building laws?

As to the moment factors in Table 3, they apply exactly to uniform loads and symmetrical span arrangements; but as loads depart from uniformity and spans from symmetry, so do these moment factors depart from true and proper values. Why "moments due to equi-distant concentrated loads are so nearly equal to uniform loads that the same moment factors would apply", is not wholly apparent. The adaptation of these moment factors to the case of different moments of inertia in the different spans is not a matter of simple facility. Neither is it clear why an attempt should be made to develop moment factors except for the supports. With the moments at the supports known, the stress distribution elsewhere becomes a simple problem, and it is only for uniform loads that these mid-span factors apply at all.

These difficulties seem to be inherent in the approach to the problem by the Theorem of Three Moments. Mr. Oesterblom himself recognizes the trouble in his statement that "clearly it is impracticable to make a table of moment factors for all possible and reasonable span combinations." He states that 2 025 combinations with 15 moment factors each would be required to

cover spans varying from 8 ft. to 30 ft. by 6-in. increments. It is many times as impracticable if, to the moment factors for all possible and reasonable span combinations, are added the moment factors for all possible and reasonable load combinations.

From another angle of approach, however, all possible and reasonable span combinations together with all possible and reasonable load combinations become subject to an extremely easy and simple treatment. A method of "unbalanced moments", developed by the writer, which is a special application of the slope deflection method, makes it possible to derive coefficients for the support moments, which are as true and "universal" as Equation (1).

Such coefficients, for the author's case of a continuous beam of three spans, with free ends and unyielding supports, are given in Fig. 6. They are in algebraic form and are expressed in terms of the "stiffness factors" of the three spans. For a constant value of the modulus of elasticity, these "stiffness factors" are simply the values obtained by dividing the moment of inertia of each span member by the span length, that is, $K = \frac{E\,I}{l}$. Fortunately, it is the relative and not the absolute value of these "stiffness factors" that is of importance and affects the stress distribution in the system. The derivation of these coefficients will not be presented herein, but the method of their use will be briefly stated and they will be applied to two problems.

The successive steps in their application are as follows:

(1) Determine the relative "stiffness factors" of the several spans.

(2) Substitute these values in Fig. 6 and obtain the numerical coefficients that are applicable to the particular problem under consideration.

(3) Consider each span of the continuous beam as a separate, individual, single span with fixed ends. Compute the end moments which would result from the loads upon the span under this assumption.

(4) Next, imagine these separate and individual fixed-end spans with their end moments to be joined and connected, to form the three-span continuous beam with free ends. Following this joining of the separate, individual, fixed-end spans, a complete re-adjustment of stress from that assumed under Step (3) occurs throughout the system. At Support (1), the free end of the continuous beam, there is nothing to resist or balance the moment which would develop there if Span 12 were the separate, fixed-end span that has previously been assumed. Consequently, this moment, unbalanced, must change and fall to zero, but in so doing it changes the stress distribution and the elastic curve of the beam in all spans. For a moment at any support to change its value implies a movement or rotation at that point, which movement and elastic adjustment carries out to the extremities of the system.

At Support (2), Spans 12 and 23 meet and are joined. Each of these spans had end moments under the assumption in Step (3). These end moments are opposite in direction—clockwise and counter-clockwise—and although they may be equal and may exactly balance one another, are generally unequal, either one side or the other being larger. It is the excess moment, the amount by which one side is greater than the other, that is unbalanced, and that, in

changing, lessening, and developing a resistance, produces a re-adjustment of stress throughout the entire system. As for the equal, opposite, and balanced moments—the full end moment of one span, and its equal and opposite

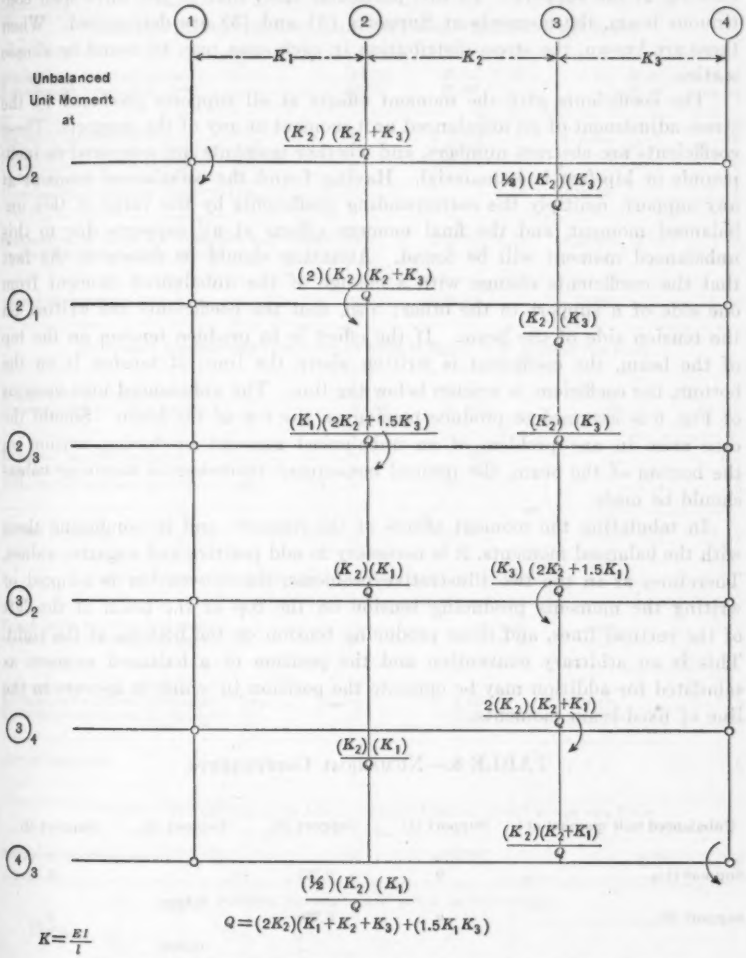

FIG. 6.—VALUES OF MOMENTS AT SUPPORTS DUE TO STRESS ADJUSTMENTS OF UNBALANCED UNIT MOMENTS.

part of the end moment of the other span—these are balanced and create no change in stress anywhere in the system, regardless of their value or magnitude. Follow a similar procedure at Supports (3) and (4).

(5) The final step is to calculate the balanced moments at each support, and the moment effects produced at each support by the several unbalanced moments. The algebraic sum of these quantities gives the actual moments existing at the supports. In this particular case, that of the three-span continuous beam, the moments at Supports (2) and (3) are determined. When these are known, the stress distribution in each span may be found by simple statics.

The coefficients give the moment effects at all supports produced by the stress adjustment of an unbalanced unit moment at any of the supports. These coefficients are abstract numbers, and whether moments are measured in inch-pounds or kip-feet is immaterial. Having found the unbalanced moment at any support, multiply the corresponding coefficients by the value of this unbalanced moment, and the final moment effects at all supports due to this unbalanced moment will be found. Attention should be drawn to the fact that the coefficients change with a change of the unbalanced moment from one side of a support to the other; also, that the coefficients are written on the tension side of the beam. If the effect is to produce tension on the top of the beam, the coefficient is written above the line; if tension is on the bottom, the coefficient is written below the line. The unbalanced unit moment of Fig. 6 is assumed to produce tension on the top of the beam. Should the case arise in any problem of an unbalanced moment producing tension on the bottom of the beam, the natural consequent inversion of resulting values should be made.

In tabulating the moment effects at the supports and in combining them with the balanced moments, it is necessary to add positive and negative values. Therefore, as in the two illustrative problems, the convention is adopted of writing the moments producing tension on the top of the beam at the left of the vertical lines, and those producing tension on the bottom, at the right. This is an arbitrary convention and the position of a balanced moment as tabulated for addition may be opposite the position in which it appears in the line of fixed-beam moments.

TABLE 8.—NUMERICAL COEFFICIENTS.

Unbalanced unit moment at:	Support (1).	Support (2).	Support (3).	Support (4).
Support $(1)_2$	0	0.365		0
Support $(2)_1$	0	0.730	0.0521	0
			0.1042	
Support $(2)_3$	0	0.270	0.1042	0

Example 1.—Consider the author's Example 1, which has a 9-ft. corridor span and two outside spans of 22 ft. 6 in. each; 67 lb. dead load; 60 lb. live load on outer spans and 80 lb. live load on corridor; and equal moments of inertia in all spans.

TABLE 9.—Numerical Solution of Example 1.

Support No.	(1)		(2)		(3)	(4)
Span length, in feet		22.5		9.0	22.5	
Uniformly Distributed Dead Load of 67 Pounds per Foot on All Spans						
Fixed beam bending moment	33.92*	33.92	5.43	5.43	33.92	33.92
Unbalanced bending moment	33.92	28.49	....	....	28.49	33.92
Balanced bending moment	0	+ 5.43	....	+ 5.43	....	0
Unbalanced moment at Support $(1)_2$	0	+12.38	....	....	− 1.77	0
Unbalanced moment at Support $(2)_1$	0	+20.80	....	....	− 2.96	0
Unbalanced moment at Support $(3)_4$	0	.....	−2.96	+20.80	....	0
Unbalanced moment at Support $(4)_3$	0	.....	−1.77	+12.38	....	0
Summation of balanced moments and unbalanced bending moment effects	0	+38.61	−4.73	+38.61	− 4.73	0
	..	− 4.73	....	− 4.73	....	..
Actual moments	0	+33.88	....	+33.88	....	0
Uniformly Distributed Live Load on Span 12 Equal to 60 Pounds per Foot						
Fixed beam bending moment	30.38	30.38	0	0	0	0
Unbalanced bending moment	30.38	30.38	0	0	0	0
Balanced bending moment	0	0	..	0	..	0
Unbalanced moment at Support $(1)_2$	0	+11.09	..	..	− 1.58	0
Unbalanced moment at Support $(2)_1$	0	+22.18	..	..	− 3.16	0
Actual moments	0	+33.27	..	..	− 4.74	0
Uniformly Distributed Live Load on Span 23 Equal to 80 Pounds per Foot						
Fixed beam bending moment	0	0	6.48	6.48	0	0
Unbalanced bending moment	0	..	6.48	6.48	..	0
Balanced bending moment	0	0	..	0	..	0
Unbalanced moment at Support $(2)_3$	0	+ 1.75	..	+ 0.67	..	0
Unbalanced moment at Support $(3)_2$	0	+ 0.67	..	+ 1.75	..	0
Actual moments	0	+ 2.42	..	+ 2.42	..	0
Uniformly Distributed Live Load on Span 34 Equal to 60 Pounds per Foot						
Actual moments	0	+33.27	..	....	− 4.74	0

* All bending moments are given in kip-inches.

Then,

$$K_1 = \frac{I}{22.5} = 1.0 \text{ (relatively)}$$

$$K_2 = \frac{I}{9} = 2.5$$

$$K_3 = \frac{I}{22.5} = 1.0$$

Substituting these values in the algebraic coefficients given in Fig. 6, the results are as shown in Table 8. The numerical solution of Example 1 is given in Table 9.

The maximum "positive" moment in Span 12 will occur when the moment at Support (2) is a mimimum. By inspection of the values found, this will occur with dead load on all spans and live load on Spans 12 and 34. With this loading the moment at Support (2) is 62 410 in-lb.

For the case of a beam of span length, l, subjected to a uniform load, w lb. per ft., with a moment, M, applied at one end, the distance of the inflection point from the end at which the moment is applied is given by Equation (36):

$$X \text{ (feet)} = \frac{2\,M}{w\,l} \qquad (36)^*$$

Between the free end and the inflection point, the stress distribution is that of a simple beam of load, w, and span, $l - x$. Therefore, the maximum positive moment in Span 12 is as follows:

Inflection point:

$$x = \frac{2 \times 62\,410}{127 \times 270} = 3.64 \text{ ft.}$$

New span:

$$22.5 - 3.64 = 18.86$$

$$M = 127 \times 355.7 \times 1.5 = 67\,600 \text{ in-lb.}$$

At Support (2) the maximum moment occurs with dead load on all spans and live load on Spans 12 and 23; combining, this is 69 570 in-lb.

At mid-span, in Span 23, the "negative" moment will be greatest with dead load on all spans and live load on Spans 12 and 34. With such loads the moments at Supports (2) and (3) are 62 410 in.-lb. Then, the moment at mid-span is,

$$62\,410 - 67 \times 81 \times 1.5 = 54\,270 \text{ in-lb.}$$

Support (3) and Span 34 are determined by symmetry. These values all agree closely with those found by the author.

Example 2.—A certain large apartment house in Seattle, Wash., has fourteen floors with spans of 22 ft. 6 in., 7 ft. 0 in., and 18 ft. 0 in.; dead load of 70 lb. per ft.; live load of 55 lb. per ft.; and relative $I_s = 5$, 1, and 5. The "stiffness factors":

$$K_1 = \frac{5}{22.5} = 0.222 = 1.6 \text{ (relatively)}$$

$$K_2 = \frac{1}{7} = 0.143 = 1.0$$

$$K_3 = \frac{5}{18} = 0.278 = 2.0$$

The numerical coefficients are as given in Table 10. The numerical solution of Example 2 is given in Table 11.

As in Example 1, the moments may be determined from Table 11 to be as follows:

* M and l must be given in same linear units.

TABLE 10.—Numerical Coefficients.

Unbalanced moment at:	Support (1).	Support (2).	Support (3).	Support (4).
Support $(1)_2$	0	0.214	0.071	0
Support $(2)_1$	0	0.429	0.143	0
Support $(2)_3$	0	0.571	0.143	0
Support $(3)_2$	0	0.114	0.628	0
Support $(3)_4$	0	0.114	0.372	0
Support $(4)_3$	0	0.057	0.186	0

Moment at Support (2), maximum..... 40 900 in-lb.
" " " (2), for maximum mid-span moment in Spans 12 and 23..... 36 100 in-lb.
Moment at Support (3), maximum..... 19 700 in-lb.
" " " (3), for maximum mid-span moment in Spans 23 and 34..... 11 600 in-lb.

The maximum mid-span moment in Span 12 is:

Inflection point:

$$x = \frac{2 \times 36\,100}{125 \times 270} = 2.14 \text{ ft.}$$

New span:

$$22.5 - 2.14 = 20.36$$

$$M = (70 + 55) \times 20.36^2 \times 1.5 = 77\,700 \text{ in-lb.}$$

and the maximum mid-span moment in Span 34 is:

Inflection point:

$$x = \frac{2 \times 11\,600}{125 \times 216} = 0.86 \text{ ft.}$$

New span:

$$18.0 - 0.86 = 17.14$$

$$M = 125 \times 17.14^2 \times 1.5 = 55\,100 \text{ in-lb.}$$

In the case of the mid-span moment in Span 23, there are moments at both ends, 36 100 and 11 600 in-lb. The excess of the larger moment at Support (2) over the smaller moment at Support (3) is 24 500 in-lb. Ordinarily, for the determination of the maximum or minimum values of the mid-span moment, this excess moment is treated precisely like the end moments in Spans 12 and 34 for finding an inflection point and new span length, etc., and, after this, a final correction is made for the smaller end moment, the value of which is applied as a constant to the results obtained. Here, however, the excess moment is so great that there is no inflection point, and no maximum or minimum value of the mid-span moment. If it is assumed that the mid-span moment is the average of the end moments, the assumption will closely approximate the truth, and any error involved will be on the safe side.

TABLE 11.—Numerical Solution of Example 2.

Support No.	(1)	(2)		(3)		(4)
Span length, in feet	22.5		7.0		18.0	
Uniformly Distributed Dead Load of 70 Pounds per Foot on All Spans						
Fixed beam bending moment	35.4*	35.4	3.4	3.4	22.7	22.7
Unbalanced bending moment	35.4	32.0	...	...	19.3	22.7
Balanced bending moment	0	+ 3.4	...	+ 3.4	....	0
Unbalanced moment at Support $(1)_2$	0	+ 7.6	...	...	— 2.5	0
Unbalanced moment at Support $(2)_1$	0	+13.7	...	...	— 4.6	0
Unbalanced moment at Support $(3)_4$	0	....	—2.2	+ 7.2	....	0
Unbalanced moment at Support $(4)_3$	0	....	—1.3	+ 4.2	....	0
Summation of balanced moments and unbalanced bending moment effects	0	+24.7	—3.5	+14.8	— 7.1	0
	..	— 3.5	...	— 7.1	....	..
Actual moments	0	+21.2	...	+ 7.7	....	0
Uniformly Distributed Live Load on Span 12 Equal to 55 Pounds per Foot						
Fixed beam bending moment	27.8	27.8	0	0	0	0
Unbalanced bending moment	27.8	27.8	0	0	0	0
Balanced bending moment	0	....	0	0	....	0
Unbalanced moment at Support $(1)_2$	0	+ 6.0	..	..	—2.0	0
Unbalanced moment at Support $(2)_1$	0	+11.9	..	..	—4.0	0
Actual moments	0	+17.9	..	..	—6.0	0
Uniformly Distributed Live Load on Span 23 Equal to 55 Pounds per Foot						
Fixed beam bending moment	0	0	2.7	2.7	0	0
Unbalanced bending moment	0	..	2.7	2.7	..	0
Balanced bending moment	0	0	...	0	..	0
Unbalanced moment at Support $(2)_3$	0	+1.5	..	+0.4	..	0
Unbalanced moment at Support $(3)_2$	0	+0.3	...	+1.7	..	0
Actual moments	0	+1.8	...	+2.1	..	0
Uniformly Distributed Live Load on Span 34 Equal to 55 Pounds per Foot						
Fixed beam bending moment	0	0	0	0	17.8	17.8
Unbalanced bending moment	0	..	0		17.8	17.8
Balanced bending moment	0	0	...	0	....	0
Unbalanced moment at Support $(3)_4$	0	..	—2.0	+6.6	....	0
Unbalanced moment at Support $(4)_3$	0	..	—1.0	+3.3	....	0
Actual moments	0	..	—3.0	+9.9	....	0

* All bending moments are given in kip-inches.

These examples have dealt with uniform loads. Any other system of loads—concentrated or variable—for which fixed-beam end moments can be calculated, can be treated with equal ease, since the method deals with unbalanced moments and not directly with the beam loads. Variable span lengths and different moments of inertia of the several spans can be handled with practically the same facility as equal spans and constant moments of inertia. The effect of settlement of supports likewise can be included in the calculations with these same coefficients in an extremely simple and ready manner.

For other than three spans, and for the cases of free ends, fixed ends, one end free, and one fixed, and of variable amounts of restraint at the ends, similar tables of coefficients can be prepared and thereby the continuous beam (continuous also in the interminable length of its required calculations) becomes simple indeed. With a simple and easy method available for calculating results, the designer can vary the moments of inertia of his members, and thereby obtain a control over the distribution of stresses in his continuous beam that heretofore has been attempted only by varying the span lengths.

While he can do this, it is to be recognized, of course, that "The Building Code", or something equally authoritative, will have its final say in the matter, and, again, the distinction between "can" and "may" is to be acknowledged and admitted—by Man, if not by Nature.

T. F. Hickerson,* M. Am. Soc. C. E. (by letter).—The writer wishes to commend Mr. Oesterblom for his interesting and valuable paper, even if it does fall far short of meeting all the conditions of practice. His Equation (1) fails to be "universal", in that it ignores restraints occurring at the beam terminals and at the connections with intermediate columns.

By means of the slope-deflection method, the writer has developed formulas and compiled a set of tables covering continuous beams of three symmetrical, two unsymmetrical, and four symmetrical, spans. Various cases of relative column stiffness have been taken into account. Three conditions of beam terminals have been assumed: (1) Ends hinged; (2) ends one-half fixed; and (3) ends fixed. All column terminals have been assumed as hinged, but the tables are readily adaptable to the case of fixed ends since the point of inflection (occurring at a known point along the column) corresponds to a hinge. Hence, the relative stiffness factor for a column with fixed ends is greater by a known amount than that for one with hinged ends.

Referring to Fig. 7, showing three symmetrical spans constructed monolithically with the columns, beam moments have been computed for Points 1, 2, and 2′ at the supports; Points *a*, *b*, and *c*, at the quarter, middle, and three-quarter points, respectively, of the first span; and Points *d*, *e*, and *f*, at the quarter, middle, and three-quarter points, respectively, of the second span. The total moments absorbed by the interior columns have also been computed and tabulated for all reasonable values of the middle-to-end span ratios. The reactions, R_1, R_2, R_3, and R_4, as well as the shears at Sections 1, 2, and 2′ have been computed.

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Similarly, the case of two unequal spans has been treated, x being the ratio of shorter to longer span. Thus, if the longer span, l_1, is taken as unity, the shorter will be $x = \frac{l_2}{l_1}$.

Continuous beams of four and five equal spans have been solved for moments and shears at numerous points. The effect of a load is practically negligible beyond two adjacent spans, hence beams continuous beyond five spans may be solved with sufficient accuracy by considering three, four, or five consecutive spans as a unit. This statement is particularly true for the usual case when the beam terminals are restrained and the columns are constructed monolithically with the beams.

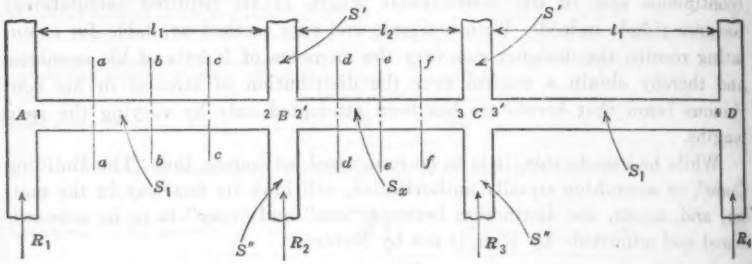

FIG. 7.

Table 12, which is typical, gives factors for the maximum bending moment at Point 2 of the second support (Fig. 7) in continuous three-span construction. Three conditions of beam terminals are considered; that is, hinged, half fixed, and fixed. Five values of the relative column stiffness, $\frac{S}{S_1}$, are considered, namely, 0, $\frac{1}{4}$, $\frac{1}{2}$, $\frac{3}{4}$, and 1.

The upper left-hand corner of Table 12, corresponding to hinged-beam terminals and no column restraint, embraces the special case treated by Mr. Oesterblom in Table 3.

The following notation has been used:

x = ratio of middle to end spans = $\frac{l_2}{l_1}$.

$S_1 = \frac{I_1}{l_1}$ = stiffness of each end beam.

$S_x = \frac{I_1}{l_2}$ = stiffness of middle beam.

S' = stiffness of upper intermediate columns.

S'' = stiffness of lower intermediate columns.

$S = \frac{S' + S''}{2}$ = average column stiffness.

ϕ = moment factor for a maximum moment, M_2, at Point 2.

TABLE 12.—Moment Factors, ϕ, for Maximum Bending Moment (M_2) at Point 2 (Fig. 7) of First Interior Support in Continuous Three-Span Construction.

Formula, $M = \phi\ w\ l^2$.

(Symmetrical span arrangement; x = ratio of middle to side spans; all moment factors are computed on basis of outside spans.)

	x.	Beam Terminals at A and D, Hinged.			Beam Terminals at A and D, One-Half Fixed.			Beam Terminals at A and D, Fixed.		
		First span loaded.	Second span loaded.	Third span loaded.	First span loaded.	Second span loaded.	Third span loaded.	First span loaded.	Second span loaded.	Third span loaded.
No Column Restraint. $\frac{S}{S_1} = 0$.	0.20	−0.105	−0.001	0.009	−0.086	−0.001	0.008	−0.067	−0.001	0.007
	0.30	−0.098	−0.002	0.011	−0.079	−0.003	0.010	−0.061	−0.003	0.009
	0.40	−0.091	−0.005	0.013	−0.073	−0.006	0.011	−0.056	−0.006	0.010
	0.50	−0.086	−0.009	0.014	−0.069	−0.010	0.012	−0.052	−0.010	0.010
	0.60	−0.081	−0.014	0.015	−0.065	−0.015	0.013	−0.049	−0.016	0.011
	0.70	−0.077	−0.021	0.016	−0.061	−0.022	0.013	−0.046	−0.024	0.011
	0.80	−0.073	−0.029	0.016	−0.058	−0.031	0.014	−0.043	−0.033	0.011
	0.90	−0.070	−0.039	0.016	−0.055	−0.041	0.014	−0.041	−0.043	0.011
	1.00	−0.067	−0.050	0.017	−0.052	−0.053	0.014	−0.039	−0.055	0.011
	1.10	−0.064	−0.063	0.017	−0.050	−0.066	0.014	−0.037	−0.069	0.011
	1.20	−0.061	−0.077	0.017	−0.048	−0.081	0.014	−0.035	−0.085	0.011
	1.30	−0.059	−0.093	0.017	−0.046	−0.097	0.014	−0.034	−0.101	0.011
	1.40	−0.057	−0.111	0.017	−0.044	−0.115	0.014	−0.033	−0.120	0.011
	1.50	−0.055	−0.130	0.017	−0.043	−0.135	0.013	−0.031	−0.141	0.010
	1.60	−0.053	−0.151	0.016	−0.041	−0.157	0.013	−0.030	−0.163	0.010
	1.70	−0.051	−0.173	0.016	−0.040	−0.180	0.013	−0.029	−0.186	0.010
	1.80	−0.050	−0.197	0.016	−0.039	−0.204	0.013	−0.028	−0.211	0.010
	1.90	−0.048	−0.223	0.016	−0.037	−0.230	0.013	−0.027	−0.238	0.010
	2.00	−0.047	−0.250	0.016	−0.036	−0.258	0.013	−0.026	−0.267	0.010
	2.10	−0.046	−0.279	0.015	−0.035	−0.288	0.012	−0.025	−0.297	0.009
	2.20	−0.044	−0.310	0.015	−0.034	−0.319	0.012	−0.025	−0.329	0.009
	2.30	−0.043	−0.342	0.015	−0.033	−0.352	0.012	−0.024	−0.362	0.009
	2.40	−0.042	−0.376	0.015	−0.032	−0.386	0.012	−0.023	−0.397	0.009
	2.50	−0.041	−0.411	0.015	−0.031	−0.423	0.012	−0.023	−0.434	0.009
Average Column Stiffness = One-Fourth Stiffness of End Beam. $S = \frac{S' + S''}{2} = \frac{1}{4} S_1$.	0.20	−0.107	−0.001	0.008	−0.088	−0.001	0.007	−0.068	−0.001	0.006
	0.30	−0.101	−0.002	0.009	−0.082	−0.002	0.008	−0.063	−0.002	0.007
	0.40	−0.096	−0.004	0.010	−0.078	−0.004	0.009	−0.059	−0.005	0.008
	0.50	−0.092	−0.007	0.011	−0.074	−0.008	0.010	−0.056	−0.009	0.008
	0.60	−0.088	−0.011	0.011	−0.071	−0.012	0.010	−0.054	−0.014	0.008
	0.70	−0.085	−0.017	0.011	−0.069	−0.018	0.010	−0.052	−0.020	0.008
	0.80	−0.083	−0.023	0.011	−0.067	−0.025	0.010	−0.050	−0.027	0.008
	0.90	−0.080	−0.030	0.011	−0.064	−0.032	0.010	−0.048	−0.035	0.008
	1.00	−0.078	−0.038	0.011	−0.062	−0.041	0.010	−0.047	−0.044	0.008
	1.10	−0.076	−0.048	0.011	−0.061	−0.052	0.010	−0.045	−0.055	0.008
	1.20	−0.075	−0.059	0.011	−0.060	−0.063	0.009	−0.044	−0.067	0.007
	1.30	−0.073	−0.070	0.010	−0.058	−0.075	0.009	−0.043	−0.080	0.007
	1.40	−0.072	−0.082	0.010	−0.057	−0.088	0.009	−0.042	−0.095	0.007
	1.50	−0.071	−0.096	0.010	−0.056	−0.103	0.009	−0.041	−0.110	0.007
	1.60	−0.070	−0.111	0.010	−0.055	−0.118	0.009	−0.041	−0.126	0.007
	1.70	−0.069	−0.127	0.010	−0.054	−0.135	0.009	−0.040	−0.144	0.007
	1.80	−0.068	−0.144	0.009	−0.053	−0.153	0.008	−0.039	−0.163	0.006
	1.90	−0.067	−0.162	0.009	−0.053	−0.173	0.008	−0.039	−0.183	0.006
	2.00	−0.066	−0.182	0.009	−0.052	−0.193	0.008	−0.038	−0.205	0.006
	2.10	−0.066	−0.202	0.009	−0.052	−0.215	0.008	−0.038	−0.228	0.006
	2.20	−0.064	−0.224	0.009	−0.051	−0.238	0.008	−0.037	−0.252	0.006
	2.30	−0.064	−0.246	0.009	−0.051	−0.262	0.008	−0.037	−0.277	0.006
	2.40	−0.063	−0.270	0.008	−0.050	−0.287	0.007	−0.036	−0.303	0.005
	2.50	−0.062	−0.295	0.008	−0.049	−0.314	0.007	−0.036	−0.331	0.005

TABLE 12.—(*Continued*).

	x.	Beam Terminals at A and D, Hinged.			Beam Terminals at A and D, One-Half Fixed.			Beam Terminals at A and D, Fixed.		
		First span loaded.	Second span loaded.	Third span loaded.	First span loaded.	Second span loaded.	Third span loaded.	First span loaded.	Second span loaded.	Third span loaded.
Average Column Stiffness = Stiffness of End Beam. $S = \frac{S' + S''}{2} = S_1$.	0.20	−0.110	−0.001	0.005	−0.091	−0.001	0.004	−0.071	−0.001	0.004
	0.30	−0.108	−0.001	0.005	−0.087	−0.002	0.005	−0.068	−0.002	0.004
	0.40	−0.105	−0.003	0.005	−0.085	−0.003	0.005	−0.066	−0.004	0.004
	0.50	−0.103	−0.005	0.005	−0.083	−0.005	0.005	−0.064	−0.006	0.004
	0.60	−0.101	−0.007	0.005	−0.082	−0.008	0.005	−0.062	−0.009	0.004
	0.70	−0.099	−0.010	0.005	−0.080	−0.011	0.005	−0.061	−0.013	0.004
	0.80	−0.097	−0.014	0.005	−0.079	−0.015	0.005	−0.060	−0.017	0.004
	0.90	−0.096	−0.018	0.005	−0.078	−0.020	0.005	−0.060	−0.022	0.004
	1.00	−0.095	−0.023	0.005	−0.078	−0.025	0.004	−0.059	−0.028	0.003
	1.10	−0.095	−0.028	0.004	−0.077	−0.031	0.004	−0.058	−0.034	0.003
	1.20	−0.094	−0.034	0.004	−0.077	−0.037	0.004	−0.058	−0.041	0.003
	1.30	−0.094	−0.040	0.004	−0.076	−0.044	0.004	−0.058	−0.049	0.003
	1.40	−0.093	−0.047	0.004	−0.076	−0.052	0.004	−0.057	−0.057	0.003
	1.50	−0.093	−0.054	0.004	−0.075	−0.060	0.004	−0.057	−0.066	0.003
	1.60	−0.093	−0.062	0.004	−0.075	−0.069	0.004	−0.056	−0.076	0.003
	1.70	−0.092	−0.071	0.004	−0.074	−0.078	0.004	−0.056	−0.086	0.003
	1.80	−0.092	−0.080	0.003	−0.074	−0.088	0.003	−0.056	−0.097	0.002
	1.90	−0.092	−0.090	0.003	−0.073	−0.099	0.003	−0.056	−0.109	0.002
	2.00	−0.091	−0.100	0.003	−0.073	−0.111	0.003	−0.056	−0.122	0.002
	2.10	−0.091	−0.111	0.003	−0.073	−0.123	0.002	−0.055	−0.134	0.002
	2.20	−0.090	−0.122	0.003	−0.073	−0.135	0.002	−0.055	−0.148	0.002
	2.30	−0.090	−0.134	0.003	−0.072	−0.148	0.002	−0.055	−0.162	0.002
	2.40	−0.089	−0.146	0.003	−0.072	−0.161	0.002	−0.055	−0.177	0.002
	2.50	−0.089	−0.159	0.003	−0.072	−0.176	0.002	−0.054	−0.193	0.002
Average Column Stiffness = Three-Fourths Stiffness of End Beam. $S = \frac{S' + S''}{2} = \frac{3}{4}S_1$.	0.20	−0.109	−0.001	0.005	−0.089	−0.001	0.005	−0.070	−0.001	0.005
	0.30	−0.105	−0.002	0.006	−0.086	−0.002	0.006	−0.067	−0.002	0.005
	0.40	−0.102	−0.003	0.007	−0.083	−0.003	0.006	−0.064	−0.004	0.005
	0.50	−0.099	−0.005	0.007	−0.081	−0.006	0.006	−0.062	−0.007	0.005
	0.60	−0.097	−0.008	0.007	−0.079	−0.009	0.006	−0.060	−0.010	0.005
	0.70	−0.095	−0.012	0.006	−0.077	−0.013	0.006	−0.059	−0.015	0.005
	0.80	−0.094	−0.016	0.006	−0.076	−0.018	0.006	−0.058	−0.020	0.005
	0.90	−0.092	−0.021	0.006	−0.075	−0.023	0.006	−0.057	−0.025	0.005
	1.00	−0.091	−0.026	0.006	−0.074	−0.029	0.006	−0.056	−0.032	0.005
	1.10	−0.090	−0.032	0.006	−0.073	−0.036	0.005	−0.055	−0.039	0.004
	1.20	−0.090	−0.039	0.005	−0.072	−0.043	0.005	−0.054	−0.047	0.004
	1.30	−0.089	−0.047	0.005	−0.072	−0.051	0.005	−0.054	−0.056	0.004
	1.40	−0.088	−0.055	0.005	−0.071	−0.060	0.005	−0.053	−0.066	0.004
	1.50	−0.088	−0.064	0.005	−0.071	−0.070	0.005	−0.053	−0.076	0.004
	1.60	−0.087	−0.073	0.005	−0.070	−0.080	0.005	−0.053	−0.088	0.004
	1.70	−0.086	−0.083	0.005	−0.069	−0.091	0.005	−0.052	−0.100	0.004
	1.80	−0.086	−0.094	0.005	−0.069	−0.103	0.004	−0.052	−0.112	0.003
	1.90	−0.086	−0.106	0.004	−0.069	−0.116	0.004	−0.052	−0.126	0.003
	2.00	−0.085	−0.118	0.004	−0.068	−0.129	0.004	−0.051	−0.140	0.003
	2.10	−0.085	−0.130	0.004	−0.068	−0.143	0.004	−0.051	−0.156	0.003
	2.20	−0.084	−0.144	0.004	−0.068	−0.157	0.004	−0.051	−0.171	0.003
	2.30	−0.084	−0.158	0.004	−0.068	−0.172	0.004	−0.051	−0.188	0.003
	2.40	−0.084	−0.173	0.004	−0.067	−0.189	0.004	−0.050	−0.206	0.003
	2.50	−0.084	−0.188	0.004	−0.067	−0.206	0.004	−0.050	−0.224	0.003
Average Column Stiffness = One-Half Stiffness of End Beam. $S = \frac{S' + S''}{2} = \frac{1}{2}S_1$.	0.20	−0.108	−0.001	0.006	−0.089	−0.001	0.006	−0.069	−0.001	0.005
	0.30	−0.103	−0.002	0.008	−0.084	−0.002	0.007	−0.065	−0.002	0.005
	0.40	−0.099	−0.004	0.008	−0.081	−0.004	0.007	−0.062	−0.004	0.005
	0.50	−0.096	−0.006	0.008	−0.078	−0.007	0.007	−0.059	−0.008	0.005
	0.60	−0.093	−0.010	0.008	−0.075	−0.011	0.007	−0.057	−0.012	0.005
	0.70	−0.091	−0.014	0.008	−0.074	−0.015	0.007	−0.056	−0.017	0.005
	0.80	−0.089	−0.019	0.008	−0.072	−0.020	0.007	−0.054	−0.022	0.005
	0.90	−0.087	−0.025	0.008	−0.070	−0.027	0.007	−0.053	−0.029	0.005
	1.00	−0.086	−0.031	0.008	−0.069	−0.034	0.007	−0.052	−0.037	0.005
	1.10	−0.085	−0.039	0.008	−0.068	−0.042	0.007	−0.051	−0.046	0.005
	1.20	−0.084	−0.047	0.007	−0.067	−0.051	0.006	−0.050	−0.055	0.005
	1.30	−0.082	−0.056	0.007	−0.066	−0.061	0.006	−0.049	−0.066	0.005
	1.40	−0.082	−0.066	0.007	−0.066	−0.072	0.006	−0.049	−0.078	0.005
	1.50	−0.081	−0.077	0.007	−0.065	−0.083	0.006	−0.048	−0.090	0.005
	1.60	−0.080	−0.088	0.007	−0.064	−0.095	0.006	−0.048	−0.103	0.005
	1.70	−0.079	−0.101	0.006	−0.063	−0.109	0.006	−0.047	−0.118	0.005
	1.80	−0.078	−0.114	0.006	−0.063	−0.123	0.005	−0.047	−0.133	0.004
	1.90	−0.078	−0.128	0.006	−0.062	−0.139	0.005	−0.046	−0.150	0.004
	2.00	−0.077	−0.143	0.006	−0.062	−0.155	0.005	−0.046	−0.167	0.004
	2.10	−0.077	−0.159	0.006	−0.061	−0.172	0.005	−0.045	−0.185	0.004
	2.20	−0.076	−0.175	0.006	−0.060	−0.189	0.005	−0.045	−0.204	0.004
	2.30	−0.076	−0.192	0.006	−0.060	−0.208	0.005	−0.044	−0.224	0.004
	2.40	−0.076	−0.211	0.005	−0.060	−0.228	0.005	−0.044	−0.245	0.004
	2.50	−0.075	−0.230	0.005	−0.060	−0.248	0.005	−0.044	−0.267	0.004

In order to illustrate the application of Table 12, let it be assumed: (1) That the two end spans are each 16 ft. and the middle span is 8 ft.; (2) that the beam terminals are one-half fixed (that is, the terminal moments are one-half the values for absolutely fixed conditions); (3) that the ratio of average column stiffness to that of the end beam is $\frac{1}{2}$; and (4) that the dead load is 1 000 lb. per lin. ft., and the live load is 2 000 lb. per lin. ft. of beam. Then, $x = \frac{8}{16} = 0.5$, and from Table 12, the values of ϕ are as follows: -0.078 for first span loaded; -0.007 for second span loaded; $+0.007$ for third span loaded; giving a total of $\phi = -0.078$.

Dead moment (full loading) $= -0.078 \times 1\,000 \times 256 = -19\,968$ ft-lb.
Live moment (first and second spans loaded) $=$
$-(0.078 + 0.007) \times 2\,000 \times 256 = \ldots\ldots\ldots\ldots -43\,520$ ft-lb.

Total maximum moment $= \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots -63\,488$ ft-lb.

Considering now the moment factors which Mr. Oesterblom gives for the outside spans, the writer believes it is better to deal with the mid-sections, for the sake of simplicity. It is true that for the case of hinged or fixed beam terminals, the moment is not quite a maximum at the mid-section (the maximum being on the left of the center for hinged ends, and on the right for fixed ends); yet when the beam terminals are one-half fixed, the maximum occurs practically at the center. In any case, it is well to have coefficients for all the reactions so that complete shear and moment diagrams may be easily drawn if desired.

What Mr. Oesterblom has to say about different moments of inertia applies equally well to the cases where restraints occur at the beam terminals and column connections. These conclusions, however, are independent of the long mathematical expressions (Equations (34) and (35)). Thus, if the moment of inertia of the middle span is only one-half that of the end spans, then its stiffness is reduced one-half, just as if its length were doubled. Hence, the proper value of x (the ratio of middle span to end span) would be doubled. The load on the middle span, however, is not doubled but remains the same, but the moment factor corresponding to the middle span loaded should be divided by 4, since the bending moment varies as the square of the span.

The illustrative example given herewith was based on the assumption of equal cross-sections throughout. If the moment of inertia of the middle span is only one-half that of the end spans, then this problem requiring the true total maximum moment (M_2) would be solved as before, except that values of the moment factor in Table 12 are found opposite $x = \frac{0.5}{\frac{1}{2}} = 1.00$. Then, the values of ϕ are: -0.069 for first span loaded; $\frac{-0.034}{4} = -0.0085$ for second span loaded; and $+0.007$ for third span loaded; giving a total of $\phi = -0.0705$.

Dead moment = $-0.0705 \times 1\,000 \times 256 =$.... $-18\,048$ ft-lb.
Live moment (first and second spans loaded) =
$-0.0775 \times 2\,000 \times 256 =$..... $-39\,680$ ft-lb.

Total maximum moment =..... $-57\,728$ ft-lb.

JOHN I. PARCEL,* M. AM. SOC. C. E. (by letter).—The writer has read this paper with much interest, and while (as the author is doubtless well aware) it does not, by any means, completely solve the problem of stress analysis in building girders continuous over three spans, nevertheless, as a step toward advancing scientific design in this field, it merits the thanks of all conscientious designing engineers.

The paper serves to emphasize again the fact that the road leading to the accurate and scientific design of structures is a long one and is beset with many difficulties. Designers need, for example, a great many more experimental data to determine how far formulas, developed for homogeneous materials, will apply to composite construction, such as reinforced concrete. They also need vastly more knowledge of the actual effects of shrinkage and time yield in such construction. Nevertheless, the fact that there are so many important factors about which designers know very little, can hardly serve as a reason for disapproving any step, however incomplete, toward rationalizing the basic methods of design, and this, the writer understands to be Mr. Oesterblom's purpose.

Since many important cases are not covered in the paper, as, for example, those in which marked asymmetry exists; in which supports are yielding; or important support moments are introduced by the action of columns or walls, it would appear that an extension of the investigation would be very desirable. On account of the tedious character of the calculations of such special cases, this would seem an excellent field for the application of the mechanical method of stress analysis by means of model structures, such, for example, as has been developed and perfected by George E. Beggs, M. Am. Soc. C. E., at Princeton University, Princeton, N. J.† It is the writer's opinion that there is a large opportunity for useful investigation, not only in the field covered by the paper, but in many allied fields as well.

R. A. CAUGHEY,‡ M. AM. SOC. C. E. (by letter).—In the presentation of this paper, Mr. Oesterblom has well emphasized the need of more care in the computation of moments in beams and girders continuous over several supports. He has also done well in presenting Table 3 and in giving a treatment for continuous beams of variable moments of inertia. In spite of all this the writer fears that this or other similar papers stop somewhat short of the goal which structural engineers would like to reach.

When continuous slabs or joists are supported by girders, the torsional rigidity of which is comparatively small, the theorem of three moments is applicable; but when beams or girders of any kind frame into columns, it seems that some other means of computing the moments should be adopted.

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† *Transactions*, Am. Soc. C. E., Vol. 88 (1925), p. 1208, *et seq.*
‡ Associate Prof. of Structural Eng., Iowa State Coll., Ames, Iowa.

This would hold in the case of the construction mentioned by Mr. Oesterblom if the girders were placed in the transverse direction of the building and joists of one type or other were placed parallel to the length. Under such arrangement one would find, by using the slope deflection theory, a large moment in the columns and, in case the columns were very heavy, a considerable decrease of positive moment in the beams or girders between them.

Some of the structural steel companies are considering very carefully the possibilities of the welded structure, and it is probably true that in a few years the bending moments in steel beams and girders may be changed materially as they should be in concrete construction at present. In both types of construction the columns will be called upon to take moment; but even then there will be a saving of materials. These facts might suggest the use of the slope deflection theory,* but any one who has ever attempted to apply that theory will agree that the necessity of anticipating the size of sections and the labor involved in the solution of large numbers of simultaneous equations will rule it out of consideration in most cases. The writer believes, however, that with certain approximations this method can be adopted, and that it will give results sufficiently close for practical use—much closer than some of the arbitrary coefficients in common use and far closer than will be obtained by the theorem of three moments. The writer has prepared a series of curves in order to apply the method rapidly enough to make it practicable.

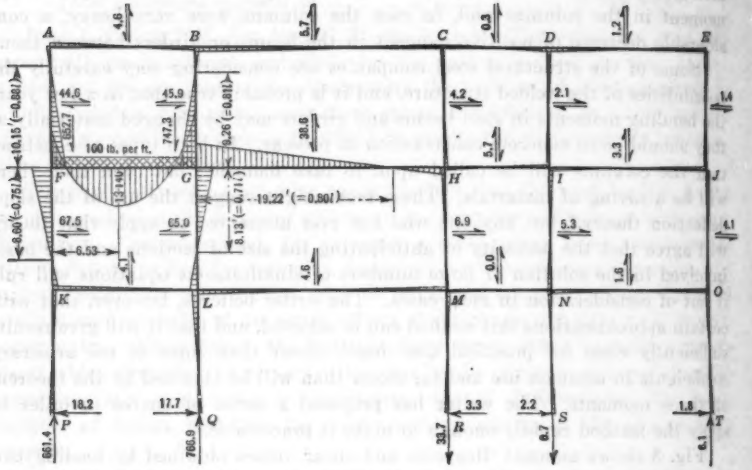
Fig. 8 shows moment diagrams and shear values obtained by loading two different panels of an unsymmetrical building frame with a uniform load.† In both cases, the effect of the load in any given panel is negligible beyond those immediately adjacent to the loaded panel. The highest value of moment outside the limits mentioned is that in Column $L\,Q$ (Fig. 8(*b*)), but very little computation is necessary to show that for any reasonable load on the girder, $G\,H$, the moment in $L\,Q$ will produce very small unit stresses in the column and that they will be maximum at the end of the column where it is stayed against buckling. This will be true generally for other members similarly located.

This would suggest dividing the structure into several parts similar to that shown in Fig. 9(*a*) and considering each part with its load separately. In the inspection of quite a large number of moment diagrams it was observed that the inflection points on columns are generally located about eight-tenths of the height above or below the floor-beams or girders. The inflection points on the horizontal members may be located from Fig. 9(*b*) which was made up from several conditions of loading on symmetrical and unsymmetrical frames. As will be noticed the location of the inflection point seems to be a function of the ratio of the lengths of loaded and adjacent spans.

The application of the slope deflection theory to such a structure becomes a very simple problem as there are only two equations to be made up and solved simultaneously. In order to simplify the derivation of equations to be

* *Bulletin No. 108*, Eng. Experiment Station, Univ. of Illinois, Urbana, Ill.

† These curves by E. E. Ebling, Assoc. M. Am. Soc. C. E., are obtained by the slope deflection method and are taken from a thesis prepared under supervision of the writer and submitted for the degree of Master of Science at the Iowa State Coll.



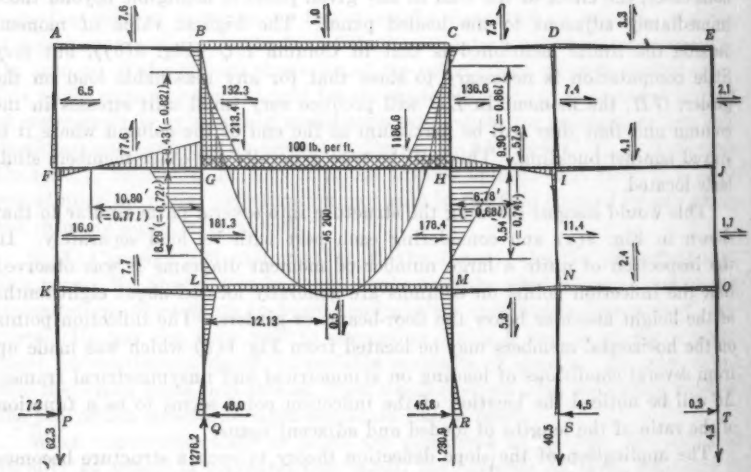


FIG. 8.—MOMENT DIAGRAMS AND SHEAR IN AN UNSYMMETRICAL BUILDING FRAME.

used, ΣK_1 will be taken as the sum of the K-values of all the members meeting the loaded beam on the left; K_2 will be the K-value for the loaded beam, and ΣK_3 the sum of the K-values of all members meeting the right end of the loaded beam. The structure can then be considered as a three-span continuous beam, $X\ A\ B\ Y$ (Fig. 9(a)), hinged at the ends, X and Y.

Applying the slope deflection equations:

$$M_{AX} = E \Sigma K_1 (3\theta_A) \quad \ldots\ldots (37)$$

$$M_{AB} = 2E K_2 (2\theta_A + \theta_B) - C_{AB} \quad \ldots\ldots (38)$$

$$M_{BA} = 2E K_2 (\theta_A + 2\theta_B) + C_{BA} \quad \ldots\ldots (39)$$

$$M_{BY} = E \Sigma K_3 (3\theta_B) \quad \ldots\ldots (40)$$

In order to have equilibrium at A and B:

$$M_{AX} + M_{AB} = 0 \quad \ldots\ldots (41)$$

and,

$$M_{BY} + M_{BA} = 0 \quad \ldots\ldots (42)$$

Substituting the values given by Equations (37), (38), (39), and (40), in Equations (41) and (42):

$$E \Sigma K_1 (3\theta_A) + 2E K_2 (2\theta_A + \theta_B) - C_{AB} = 0 \quad \ldots\ldots (43)$$

and,

$$E \Sigma K_3 (3\theta_B) + 2E K_2 (\theta_A + 2\theta_B) + C_{AB} = 0 \quad \ldots\ldots (44)$$

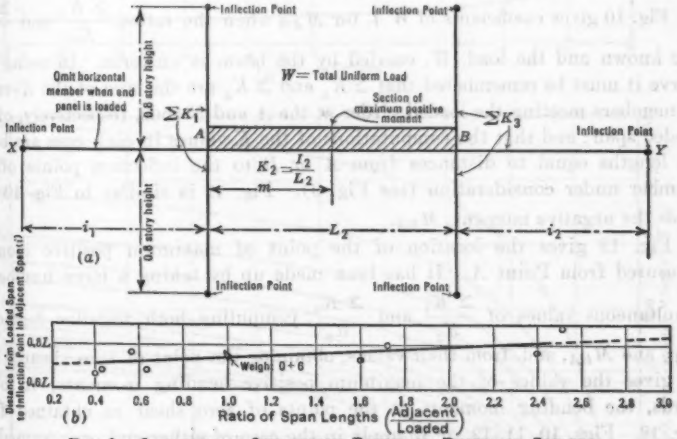


FIG. 9.—LOCATION OF INFLECTION POINTS IN BEAMS OR GIRDERS.

Assuming a symmetrical loading for $A\ B$ (Fig. 9(a)), in which case, $C_{AB} = C_{BA}$; and solving Equations (43) and (44) simultaneously, values of θ_A and θ_B are obtained which, when substituted in Equations (38) and (39), give values of M_{AB}* and M_{BA} as follows:

$$M_{AB} = -3 \Sigma K_1 C \left(\frac{2K_2 + \Sigma K_3}{4K_2 (\Sigma K_1 + K_2) + \Sigma K_3 (3 \Sigma K_1 + 4K_2)} \right) \quad \ldots\ldots (45)$$

*It will be noted that M_{AB} and M_{BA} have opposite signs. For the method of determining signs and applying the slope deflection method generally, *see Bulletin No. 108*, Eng. Experiment Station, Univ. of Illinois.

and,

$$M_{BA} = 3 \Sigma K_3 C \left(\frac{2 K_2 + \Sigma K_1}{4 K_2 (\Sigma K_1 + K_2) + \Sigma K_3 (3 \Sigma K_1 + 4 K_2)} \right) \ldots (46)$$

It is obvious that both M_{AB} and M_{BA} produce tension in the top fibers of the horizontal member, $A B$, ordinarily referred to as "negative".

Positive moment near the center line of $A B$ may be calculated with little difficulty, by first computing, by means of "moment transfer", the shear at one or both ends of $A B$ and then finding the point of zero shear or maximum moment and transferring the moment from one of the supports.

As has been suggested by Mr. Ebling, the moments carried by members of adjacent panels may be obtained by apportioning the moments, M_{AB} and M_{BA}, in such a way that the moment carried by each of the members meeting the end of the loaded girder is proportional to its K-value. These latter moments act to oppose the moments, M_{AB} and M_{BA}, thus holding the joint in equilibrium.

Explanation of Curves for Computing Moments.—In actual practice it may be necessary, in order to secure the proper sections for the beams and columns of a building, to design preliminary sections and revise them at least once and probably oftener. This would involve a large amount of computation and consume quite a little time. In order to overcome these difficulties, the writer has prepared curves which are practically self-explanatory.

Fig. 10 gives coefficients of $W L$ for M_{AB} when the ratios, $\frac{\Sigma K_1}{K_2}$ and $\frac{\Sigma K_3}{K_2}$, are known and the load, W, carried by the beam is uniform. In using this curve it must be remembered that ΣK_1 and ΣK_3 are the sum of the K-values of members meeting the loaded girder at the A and B ends, respectively, of the loaded span; and that the computations of the K-values in each case are based on lengths equal to distances from A or B to the inflection points of the member under consideration (see Fig. 9). Fig. 11 is similar to Fig. 10 and gives the negative moment, M_{BA}.

Fig. 12 gives the location of the point of maximum positive moment measured from Point A. It has been made up by taking a large number of simultaneous values of $\frac{\Sigma K_1}{K_2}$ and $\frac{\Sigma K_3}{K_2}$, computing both negative moments, M_{AB} and M_{BA}, and, from their values, obtaining the point of zero shear. Fig. 13 gives the values of the maximum positive bending moments; in other words, the bending moments at the points of zero shear as obtained from Fig. 12. Figs. 10, 11, 12, or 13 apply in the case of either end or intermediate spans.

Illustrative Problem.—Compute the bending moments and shears in a building frame, a part of which is shown by Fig. 14, for a uniform load of 2 000 lb. per lin. ft. of girder.

Moment Diagrams.—First, load Span $A B$ and disregard all the structure to the right of Point C. The ratio of adjacent to loaded span length is $\frac{9}{22.5} = 0.4$; then, from Fig. 9 (b), the point of inflection of Span $B C$ is found to be $0.72 \times 9 = 6.48$ ft. to the right of Point B. The points of inflec-

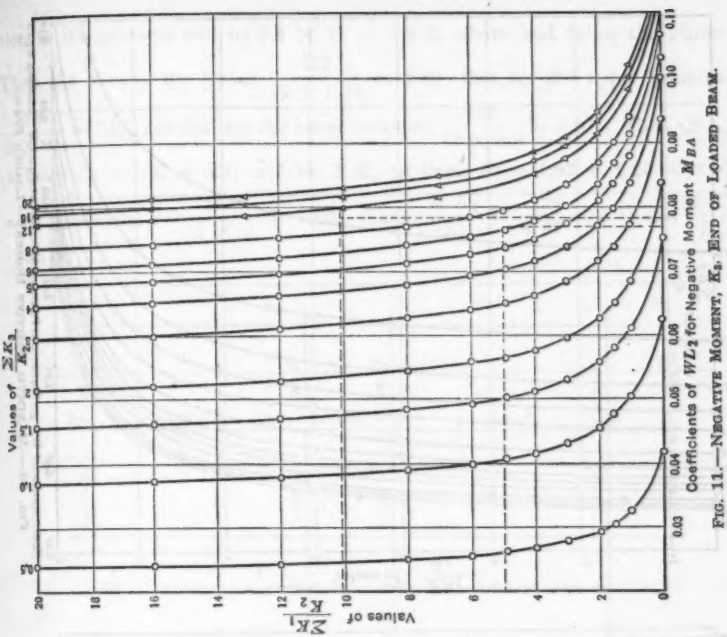


FIG. 11.—NEGATIVE MOMENT, K_2, END OF LOADED BEAM.

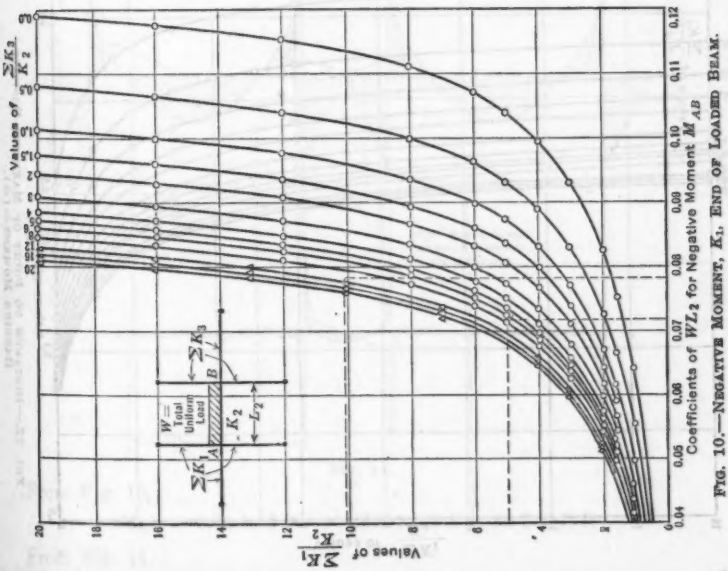


FIG. 10.—NEGATIVE MOMENT, K_1, END OF LOADED BEAM.

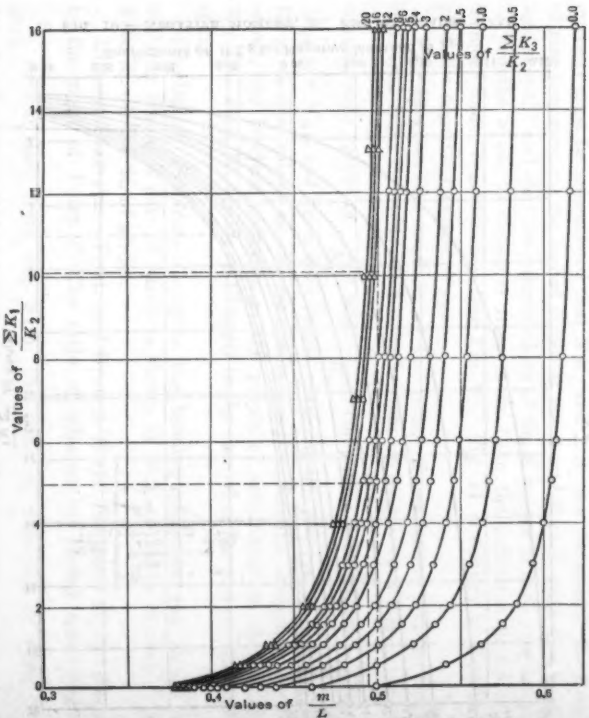


FIG. 12.—DISTANCE TO POINT OF MAXIMUM POSITIVE BENDING MOMENT—(m).

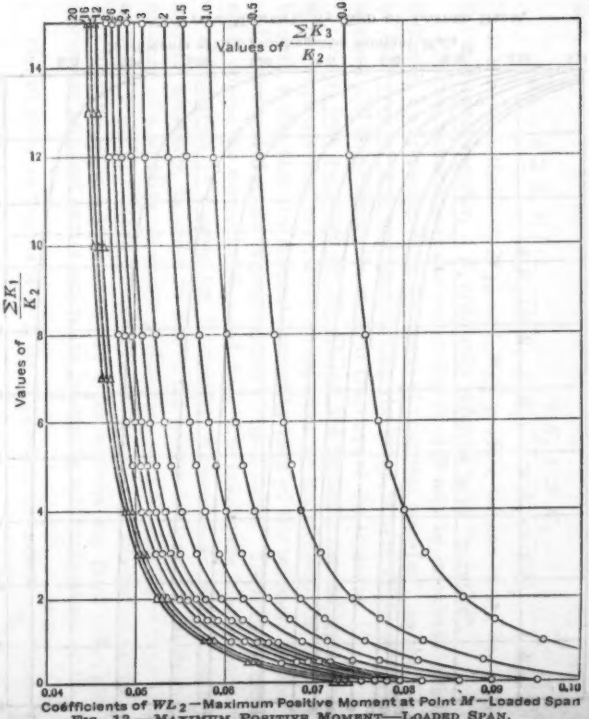


FIG. 13.—MAXIMUM POSITIVE MOMENT—LOADED SPAN.

tion in the columns will be $0.8 \times 11 = 8.8$ ft. above and below the girder. Then, the K-value for BC is $\frac{173}{12 \times 6.48} = 2.23$; that for the upper columns, $\frac{208}{12 \times 8.8} = 1.97$; and that for the lower columns, $\frac{630}{12 \times 8.8} = 5.97$. Then, ΣK_1 (at Point A) $= 1.97 + 5.97 = 7.94$; ΣK_3 (at Point B) $= 1.97 + 2.23 + 5.97 = 10.17$; and $K_2 = \frac{173}{12 \times 9} = 1.60$. Then, $\frac{\Sigma K_1}{K_2} = \frac{7.94}{6.67} = 1.19$ and $\frac{\Sigma K_3}{K_2} = \frac{10.17}{6.67} = 1.52$.

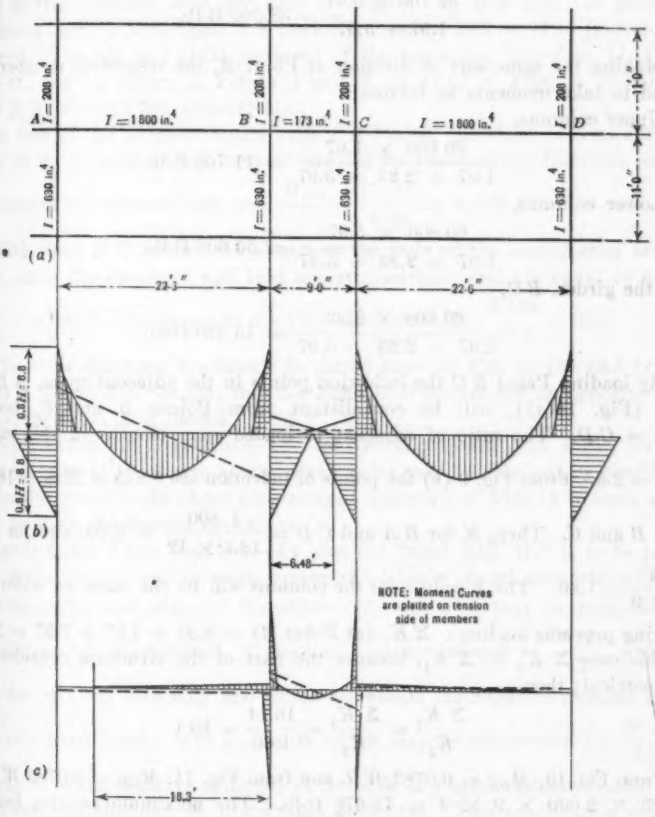


FIG. 14.

From Fig. 10,

$$M_{AB} = 0.05 \times 2\,000 \times 22.5 \times 22.5 = 50\,700 \text{ ft-lb.}$$

From Fig. 11,

$$M_{BA} = 0.06 \times 2\,000 \times 22.5 \times 22.5 = 60\,800 \text{ ft-lb.}$$

From Fig. 13,
Maximum positive moment $= 0.071 \times 2\,000 \times 22.5 \times 22.5 = 71\,800$ ft-lb.

From Fig. 12 the maximum bending moment occurs $0.49 \times 22.5 = 11.02$ ft. from Point A. (See Fig. 14(b).) Dividing the moment opposing AB between the upper and lower columns in proportion to the K-values of the columns, the moment in the upper columns is,

$$\frac{50\,700 \times 1.97}{1.97 + 5.97} = 12\,570 \text{ ft-lb.}$$

and that in the lower columns is,

$$\frac{50\,700 \times 5.97}{1.97 + 5.97} = 38\,000 \text{ ft-lb.}$$

Making the same sort of division at Point B, the respective members are found to take moments as follows:

Upper columns,

$$\frac{60\,800 \times 1.97}{1.97 + 2.23 + 5.97} = 11\,700 \text{ ft-lb.}$$

Lower columns,

$$\frac{60\,800 \times 5.97}{1.97 + 2.23 + 5.97} = 35\,600 \text{ ft-lb.}$$

and the girder, $B\,C$,

$$\frac{60\,800 \times 2.23}{1.97 + 2.23 + 5.97} = 13\,320 \text{ ft-lb.}$$

By loading Panel $B\,C$ the inflection points in the adjacent spans, $A\,B$ and $C\,D$ (Fig. 14(a)), will be equi-distant from Points B and C, because $A\,B = C\,D$. The ratio of adjacent to loaded span lengths in this case is $\frac{22.5}{9} = 2.5$. From Fig. 9 (b) the points of inflection are $0.815 \times 22.5 = 18.3$ ft. from B and C. Then, K for $B\,A$ and $C\,D = \frac{1\,800}{18.3 \times 12} = 8.20$, and for $B\,C$, $\frac{173}{12 \times 9} = 1.60$. The K-values for the columns will be the same as when considering previous loading: ΣK_1 (at Point B) $= 8.20 + 1.97 + 5.97 = 16.14$. In this case $\Sigma K_3 = \Sigma K_1$, because the part of the structure considered is symmetrical; then,

$$\frac{\Sigma K_1}{K_2} = \frac{\Sigma K_3}{K_2} = \frac{16.14}{1.60} = 10.1$$

From Fig. 10, $M_{BC} = 0.0783\ W\,L$ and from Fig. 11, $M_{CB} = 0.0783\ W\,L = 0.0783 \times 2\,000 \times 9 \times 9 = 12\,670$ ft-lb. The maximum positive bending moment at mid-span (Fig. 13) in this case is, $0.047 \times 2\,000 \times 9 \times 9 = 7\,610$ ft-lb.

Apportioning the resisting moments, M_{BC} and M_{CB}, between adjacent members at Points B and C,

$$M_{BA} = M_{CD} = \frac{12\,670 \times 8.2}{8.2 + 1.97 + 5.97} = 6\,430 \text{ in-lb.}$$

$$M \text{ (upper columns)} = \frac{12\,670 \times 1.97}{8.2 + 1.97 + 5.97} = 1\,547 \text{ in-lb.};$$

and,

$$M \text{ (lower columns)} = \frac{12\,670 \times 5.97}{8.2 + 1.97 + 5.97} = 4\,680 \text{ in-lb.}$$

Shear Diagrams.—The computation of shear for these cases is very simple. In the loaded spans the point of zero shear is the point of maximum bending moment; and, since the uniform load is known, the inclination of the line representing the variation in shear is known and the shears at the ends can be obtained by multiplying the length of right and left segments of the loaded beam by the uniform load per unit of length. In this case the point of maximum bending with Span $A\,B$ loaded is $0.495 \times 22.5 = 11.15$ ft. from the left end. The left and right segments of the beam are, therefore, 11.15 and 11.35 ft., and the shears at Points A and B are $11.15 \times 2\,000 = 22\,300$ and $11.35 \times 2\,000 = 22\,700$, respectively.

In case of the adjacent spans with $A\,B$ loaded, the shear is constant and equal to the maximum end moment divided by the distance from the end to the point of inflection; in this case, $\frac{M_{BC}}{i} = \frac{13\,200}{6.48} = 2\,060$.

With Span $B\,C$ loaded, the shears at the ends of the loaded span are the same, since the structure and load are symmetrical, and are equal to $4.5 \times 2\,000 = 9\,000$ lb., the shears in the adjacent spans being $\frac{6\,430}{18.3} = 355$.

The shear diagrams are drawn in dotted lines on Fig. 14 (*b*) and (*c*) for Spans $A\,B$ and $B\,C$ loaded; that for Span $C\,D$ loaded will be opposite hand to that for Span $A\,B$ loaded.

Combination of Diagrams.—The maximum moment and shear for any given member of the frame at any given point may be obtained by combining the results given in the shear and moment diagrams of Fig. 14 in such a way as to produce the desired maximum.

Loads Other Than Uniform.—In case the "total load, W," is to be placed on a given span as one or more concentrated loads placed symmetrically, the moments, M_{AB} and M_{BA}, of Equations (45) and (46) may be computed by substituting for the constant, C, another constant instead of $\frac{W\,L}{12}$, which is used for uniform load only and which represents the negative moment at the ends of a fixed beam. The constants which may be substituted for $\frac{W\,L}{12}$ are the values of the negative moments at the ends of a symmetrically loaded fixed beam carrying the load, W, distributed as desired. This substitution suggests computing first the ratio of the value of C with the new loading condition to $\frac{W\,L}{12}$ and multiplying the values of the negative moments at the ends of the loaded girder found from Figs. 10 and 11 by the ratio thus obtained. With

these moments computed, all necessary moments and shears may be calculated in a manner similar to that used in the example given for uniform loads.

As to the accuracy of the results obtained by this method and by the curves submitted, the writer has checked the results obtained by Mr. Ebling and has found that in four loading conditions in a very unsymmetrical building, the moments in the girder of the loaded panel (at ends and near mid-span) obtained by the curves have checked Mr. Ebling's slope deflection results with a maximum error of 7.6 per cent. The moments in question taken from the curves have occurred, of course, on the working line of the building frame, which is, in this case, on the center line of the columns. The moment at the face of the column would be actually somewhat less than that at the center line, and since higher unit stresses are usually allowed in a monolithic building frame in sections of members adjacent to supports, it would seem that the 7.6% variation mentioned, would cause no trouble.

The check has not been as good on members in which the moment has been obtained by proportion between the values of K, because the maximum error in one case—and in only one case—reached 14 per cent. All checks, however, have been close enough in the writer's opinion to warrant the use of these curves or similar ones in place of arbitrary coefficients or even the three-moment equation, when there is a possibility of the columns and beams (or girders) forming a rigid frame. This opinion is based on the fact that the allowable increase in unit stresses at the ends of members, together with a decrease in moment at the face of the members into which they frame, as compared to the moment at the center line of the latter, would absorb, in practically all cases, a difference as great as 14%; in some cases much more.

JOHN B. SNETHLAGE,* ASSOC. M. AM. SOC. C. E. (by letter).—The author has certainly given a clear idea of how he thinks continuous beams over three spans ought to be designed, and the results are most efficiently laid down in tables which cover a sufficient number of span varieties to warrant usefulness. Little is to be added to his general treatment of the problem. It is well known that normally the conditions at the supports are not as he assumes them to be and that restraining moments at the ends counteract the theoretical ones.

The author makes a severe attack on conventional moment factors, of building codes, joint committee regulations, etc., and attempts to illustrate their shortcomings with an example which seems to make their use prohibitive.

When his method gives the correct negative moment as —4 490 ft-lb. in the center span, and the conventional moment-factor method, according to him, calls for + 940 ft-lb. (see Table 4), the writer can hardly believe that there can be compensating features to prevent failure. In the writer's opinion the author does not apply these moment factors as they were intended.

As shown in Table 4, the correct maximum moments for the conditions given, are: + 5 740 ft-lb. for outer spans; —4 490 ft-lb. for center span; and —5 770 ft-lb. for center supports.

* Civ. and Hydr. Engr., Hawthorne, N. J.

By applying the conventional moment factors, a positive moment is found in the outer spans equal to $\frac{1}{10} \times 127 \times 21.75^2 = 6\,000$ ft-lb., and at the center supports, not some kind of an average as stated by the author, but a negative moment is used of the same absolute value as the positive moment, or $-6\,000$ ft-lb. (See Fig. 15.)

This negative moment prevails over the entire center span only to be reduced by the positive moment of the center span, or $\frac{1}{12} \times 147 \times 8.3^2 = 850$ ft-lb. Therefore, this span must be reinforced for a negative moment, in the middle, of $6\,000 - 850 = 5\,150$ ft-lb.

Comparison of these figures with Table 4 shows that the moment factor method requires 4% more steel at the center supports and 4.5% more steel at the outer spans, against which hardly any objection can be raised. On the center span, however, this waste is 15% which seems excessive; but in reality it is of little importance, as that negative moment increases very rapidly toward the center supports.

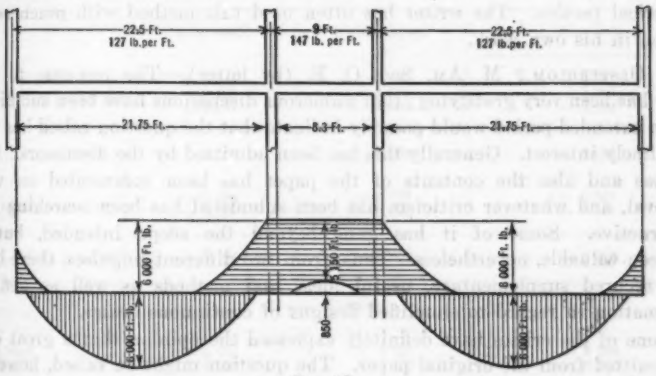


FIG. 15.

Moreover, in detailing it will not be found possible to follow the fluctuation accurately by decreasing and again increasing the amount of steel over so short a distance as 9 ft., and most likely the steel from the negative moment at one center support will be continued over the entire center span to the other support.

This demonstrates clearly what the writer believes to be the proper application and true interpretation of the use of the conventional moment factors of building codes, joint committee regulations, etc., and although by no means ideal, they certainly do not deserve such unreasonable criticism.

T. D. MYLREA,* M. AM. SOC. C. E. (by letter).—The author is certainly justified in his criticism of the practice of using arbitrary moment coefficients in many cases to which they do not apply, and is to be commended for his

* Prof. of Building Constr., Carnegie Inst. of Tech., Pittsburgh, Pa.

efforts toward simplifying the methods of finding the true moment coefficients in a three-span continuous beam. The labor involved and the time consumed in computing the true coefficients by the usual methods are undoubtedly responsible for the present reliance on arbitrary coefficients. As the author points out, this is particularly true in the usual analytic solution when a beam of more than three spans is encountered. In that case, the graphical solution of Messrs. Nishkian and Steinman* may prove more expeditious.

The analytic solution, however, may be approached from a standpoint distinctly different from that taken by the author, and one which permits of the solution of multi-span beams with comparative ease. The kernel of this method is suggested in Mr. Hadley's discussion. It consists of distributing throughout the beam the unbalanced, or excess, fixed-end moments. The method Mr. Hadley proposes is not entirely new, however, for the solution by distributing unbalanced moments has been taught by Hardy Cross, M. Am. Soc. C. E., for several years.

Furthermore, Professor Cross' method† is applicable to beams of any number of spans, as well as to continuous bents, and the labor involved is small. Moreover, by this method there is no need to schedule equations and certain numerical factors. The writer has often used this method with much satisfaction in his own work.

I. Oesterblom,‡ M. Am. Soc. C. E. (by letter).—The response to this paper has been very gratifying; that numerous discussions have been submitted for an extended period would possibly indicate that the question raised has real and timely interest. Generally this has been admitted by the discussors. The purpose and also the contents of the paper has been commented on with approval, and whatever criticism has been submitted has been searching and constructive. Some of it has gone beyond the scope intended, but it has been valuable, nevertheless. Thus, from two different impulses, there have been secured supplementary, useful ideas and methods as well as definite information in regard to simplified designs of continuous beams.

Some of the critics have definitely expressed the opinion that a great deal was omitted from the original paper. The question might be raised, however, if a very complete paper on the subject would not have been an unfortunate contribution. It would have reduced the opportunity of supplementary contributions from interested engineers, but even worse than that, it would have entered into the more general field of stresses and strains in elastic frames, already so thoroughly and competently treated by a legion of highly qualified authorities.

It was the purpose of the paper—unfortunately it seems to have been insufficiently emphasized—to reduce to easily workable form some of the very complicated combinations of formulas for continuous beams, so that reasonably correct moment factors might be quickly ascertained for the most common cases of simple continuity.

* "Moments in Restrained and Continuous Beams by the Method of Conjugate Points", by L. H. Nishkian and D. B. Steinman, Members, Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 90 (1927), p. 1.

† Available in mimeographed copies for the students of the Univ. of Illinois.

‡ Structural Engr., St. Louis, Mo.

Perhaps the examples given were presented in a manner which seemed to contradict the purpose of speed in execution. By neglecting variability of load the elements of each moment factor may be summed up into one. This, in most cases, would be warranted either by fact or a very close approximation. The examples should be left as shown, however, because they give very completely and accurately all that is essential for simple cases of continuity within the limits outlined. They are typical of cases with most extensive variations.

For the routine work in a designing office, the computation form shown in Table 13 would possibly serve best. (See Example 1.)

TABLE 13.—FORM FOR COMPUTATION OF MOMENTS.

	Bending moments.		
Dead load	$67\ \phi \times 22.5^2 = 34\ 000\ \phi$		
Live load	$60\ \phi \times 22.5^2 = 30\ 500\ \phi$		
	MOMENT FACTORS (ϕ).		
	Outer span.	Middle span.	Supports.
Dead load	+ 0.084 − 0.003 + 0.007	− 0.039 + 0.015 − 0.039	− 0.091 − 0.005 + 0.013
Total	+ 0.088	− 0.063	− 0.083
Live load	+ 0.084 + 0.007	− 0.039 − 0.039	− 0.091 − 0.005
Maximum	+ 0.091	− 0.078	− 0.096
	TOTAL MOMENTS.		
Dead load	+ 2 980	− 2 130	− 2 800
Live load	+ 2 760	− 2 360	− 2 930
Total	+ 5 740	− 4 490	− 5 730

A small error occurs, for the support moment only, and this is due to taking the live load at 60 lb. instead of 80 lb. for the center span. The error is less than 1 per cent.

Rapid pencil notations obviously are sufficient, with the multiplications done by slide-rule. The simplest problems thus can be solved in less than 5 min. and the more complicated in less than 10 min.

It has become necessary to emphasize the simplicity and the time element, because so many "simplified methods" have been suggested in the discussion. These are all excellent, and will be referred to in the proper place, but they generally fail to take the final step of producing directly applicable moment factors. While there must be problems which require a great deal of care and

a corresponding amount of time, because they are complicated by supplementary restraints and variabilities, the majority of cases should be, and are, elementary enough to be solved by exceedingly rapid methods. Anything less than this would merely induce, as it has in the past, many rough approximations with corresponding inaccuracies. This is an economic problem; if the cost of designing should run too high, it will be turned into a game of chance.

This is not discounting the many excellent contributions which are of a supplementary nature. They are truly supplementary, because they show how the problems should be solved when they fall beyond the scope of the paper.

Indeed it is difficult to imagine a more graceful, ingenious, and efficient method than that of Professor Cross, or the equally ingenious one by Professor Beggs, both of which have a general application and each its special, although somewhat overlapping, field of usefulness. Professor Cross' method contains a very slight approximation, but it is entirely too small to consider in view of the exceedingly valuable time-saving procedure.

Professor Cross' method is especially recommended for all cases of continuity, that are not soluble, or should not be solved by means of ready-made moment factors. Professor Beggs' method requires a set of instruments and the manipulation of celluloid models. With the complete equipment at hand its usefulness would be unlimited.

It is particularly gratifying, therefore, that these two methods were introduced by Professors Mylrea and Parcel, but while this acknowledgment is made with gratitude for the reference and admiration for the methods, it is only fair to state that they offer neither negation nor obsolescence for the writer's contribution.

The paper is elementary in its nature; but none the less it was not intended as a text for junior students. It assumed that the reader would know that there were additional restraints; it also assumed that he would know when the use of the simplified formulas and tables would not be permissible. If the writer took this knowledge for granted erroneously, it has been made very clear by the discussion that there are limits beyond which the formulas and factors should not be used.

As soon as one has to deal with more than three spans, or additional restraints from supporting or supported elements, all possibilities for considerable simplification vanish.

Fortunately, however, the vast majority of continuous beams are of three spans and are freely or almost freely supported. The tables and formulas in the paper are, therefore, an offering to those engineers—and their assistants—who thoroughly know their elasticity problems, but who are none the less looking for a rapid and accurate solution of those that confront them most frequently.

In a sense it may be considered an accident that, from the many methods possible, the three-moment theorem was selected as a basis for the paper. It is more than that, however. It is a tribute to a most graceful and accurate solution of an old problem, which has lost nothing by having attained a classical distinction.

From another point of view it stands out with compelling clarity that the one thousand and one (more or less) mathematical masterpieces analyzing stress and strain in elastic frames have had the practical effect of beclouding the elementary issues and setting up—instead of desired simplicity and accuracy—a chimera of faith, hope, and fantasy, which, unfortunately, has been set in a golden frame by committees of authorities and experts. This evil is international, but, curiously enough, it is subject to worship, nationally.

From this viewpoint the method introduced by Professor Cross is particularly significant. From the mass of formulas that are always necessary he has extracted all the essential factors and eliminated others which have insignificant effects on the result and merely tend to confuse the designer in his mathematical operations.

The question has been raised as to what extent brick walls would serve as a restraint for the floor construction. Seemingly, there can be no mathematical answer. To one who has spent a great deal of time in the field, the restraint appears to be generally small. Sometimes, the walls may be heavy, the brick especially good, and the mortar excellent—any one or all of these—and restraint then may be considered. It certainly should be considered whenever it may introduce an element of weakness, but this is seldom the case with the supported three-span elements. More often the restraint is very small, the reduction of moments will likewise be small, and the proposed formulas and factors will be sufficient. In general, the smaller and simpler structures analyzed in this paper are by far in the majority.

The temperature stresses in three-span construction are largely insignificant, but allowance for them can be made by the usual method, as soon as fundamental proportions have been once established. Mr. Way has made reference to the possible dangers, but his examples do not seem to be convincing. Generally, structures are more protected than he has assumed. The writer agrees that shrinkage stresses in concrete frames are of definite significance, but, unfortunately, little is known of the extent and magnitude of these colloidal changes, as measured by prevailing moisture conditions in their relation to the impermeability of the mass. Fortunately, these changes are of small significance in temperate climates; but in the tropics they are very destructive.*

The writer has made a general study of this subject, but there is a great deal of specific investigation yet to be done along these lines, before colloidal stresses may be definitely predetermined. As the shrinkage stresses, in their nature, are colloidal, Mr. Way is due special thanks for pointing out their significance. Greater danger, however, is to be found in longitudinal changes of beams and girders than in columns, as noted; this is on account of their relative positions and also because setting stresses in columns are confined to one story at a time, and are not cumulative.

From the viewpoint of bringing out the necessity of further investigations along these lines, the discussion has been especially fruitful. Some discussors have wrongly assumed that the paper was intended as a guide solely for

* *Journal*, Inst. of Engrs. (India), April, 1925, p. 52.

concrete work. As a matter of necessity, the concrete designers were long ago compelled to pay attention to continuity in a frame, but steel designers also soon had to "follow suit." To assume a free connection for a girder in computing stresses due to load—and that the same connection was elastic for wind effect—was indeed absurd.

Professor Morris' contribution is, therefore, especially welcome. He has expressed the desirability for a clear statement of fundamentals and for reserving for special discussion the peculiarities of individual materials. Some discussion has been unnecessarily critical for want of clarity on this point; on the other hand, there have been most excellent supplements with many useful methods, formulas, tables, etc., added to the original paper. The discussions of Messrs. Hyde, Wilder, Hickerson, and Caughey are very valuable from this point of view.

Messrs. Wessman, Way, and Snethlage seem to agree that the paper has been unduly critical in regard to conventional practice. Mr. Wessman's experience evidently is limited to more recent years, during which better design methods have been making remarkable progress. His connections, likewise, have been fortunate.

The writer's experience, of being forced to battle against antiquated methods, has been duplicated by many. He has heard statements from progressive and prominent men of experience, that younger engineers have turned cynics because they have often been forced by older men in authority —the commercially minded or the "stick in the mud" variety—to do what they knew to be incorrect.

Mr. Way seems to have been fortunately placed. His cheerful statement is, therefore, gratifying, but it does not cover the full field. That so many, who are consistently at fault, have failed to offer any contributions is significant as contrasted with so much constructive discussion.

The discussion by Mr. Snethlage has special interest, because he admits that there is a "true interpretation of the use of the conventional moment factors" for variable spans, whereas no such use was ever intended by the members of the Joint Committee on Specifications for Concrete and Reinforced Concrete or bodies of equal authority. There could be no better demonstration that confused ideas prevail and that greater clarity is needed. His short method, however, is among the best; but, unfortunately, it is very far from being generally used. It suffers also from an element of uncertainty in that the designer never knows how far from danger he really is, or how much material is being used wastefully.

Professor Mylrea makes reference to the excellent graphical methods of Messrs. Nishkian and Steinman. There seems to be one error in the presentation, which must be guarded against in use—the necessity for transforming load values when spans are changed, to compensate for variability of section.

There is an especially clear statement of the economic significance of the writer's formulas and tables from Mr. Hadley. He dwells again on the importance of repeated typical designs, and in a much clearer manner than the writer did in the paper. Possibly several hundred similar beams and

joists may be built. Error and waste are, therefore, both multiplied; but, likewise, the economies are worth while when the calculation is done carefully and correctly.

There must be some inducement, however, to designing even the numerous small structures correctly, and no inducements could be more attractive than simplicity of method, accuracy of results, and economy of materials. Therein lies the justification for this paper. It is to be hoped, however, that the tables may soon be supplemented and ultimately supplanted by experimental moment factors obtained by Professor Beggs' method.

Mr. Hadley refers to the writer's statement about equi-distant concentrated loads and demands a more complete explanation. Reference is obviously first and foremost to the many so-called joist designs, where complete uniformity may be assumed. It should be noted, however, that even quarter-point and half-point loading will increase the moments only a very small fraction. Hence, the tables may be used—judiciously, of course—even for concentrated loads within fair limits.

Thanks to Professor Hickerson it has been possible to verify and correct the numerical factors in Table 3.

The Joint Committee on Specifications for Concrete and Reinforced Concrete has found a defender in Mr. Way. The excellent work by the Committee cannot be questioned. It was merely its attitude of inconsistency in regard to proposed moment factors which was deplored. Either the subject should have been treated more fully, or it should have been treated in general terms only. To select the simplest problems and to treat these in great detail, while at the same time omitting completely all the more difficult ones, can only confuse those who specially need the guidance of the report. Because of frequent errors of design due to misinterpretation of the moment factors specified by the Joint Committee, this criticism seems pertinent.

Erroneous application of conventional moment factors to a series of spans of different lengths, is common—almost universal. It is Mr. Way's belief that the impossibility of creating "an accurate design" has made it necessary for Code Committees to adopt empirical factors. The effect of these empirical factors seems to have been the reverse of what was intended, namely, to induce unqualified people to undertake difficult designs by the simple expedient of letting these very limited factors do serious work in the dark hinterland beyond the borders of their own ignorance.

This obviously raises the question as to the true function, as well as the limitations, of a legal "code" or a "committee report" (which, in advance of full knowledge on a subject, anticipates the code). It is not the writer's intention to dwell largely on this, but simply to raise again the point that no committee should endeavor to establish by code or rule simple matters that pertain to gravity, or anything experimentally determinable therefrom. A code should cover only matters of judgment where there is a chance of judicious selection and where the mature judgment of especially selected men will benefit the community as a whole.

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Paper No. 1715

THE O'SHAUGHNESSY DAM AND RESERVOIR*

By John H. Gregory,† C. B. Hoover,‡ and C. B. Cornell,§
Members, Am. Soc. C. E.

With Discussion by Messrs. J. Ernest Carman, B. F. Jakobsen, W. H. Dittoe and G. Gale Dixon, and John H. Gregory, C. B. Hoover, and C. B. Cornell.

Synopsis

This paper describes the design and construction of a dam and reservoir built to increase the storage of raw water for the water supply of the City of Columbus, Ohio. The masonry portion of the dam is of the overflow gravity type, the spillway section, 879 ft. in length between end abutments, being spanned by a 12-arch reinforced concrete bridge. The crest of the spillway is 64 ft. above low water in the river and has a discharge capacity of 110 000 cu. ft. per sec. The reservoir, formed by the construction of the dam, is 8 miles in length, has a maximum width of 1 900 ft., and a total available storage capacity of 5 341 000 000 gal. The total cost of the dam and reservoir was $2 220 080.

Introduction

The water supply of the City of Columbus, Ohio, is taken from the Scioto River. In 1908 the city completed a five-year program of important waterworks improvements which included the construction of a water purification and softening works and pumping station adjacent to the Scioto River, about 2 miles from the center of the city, and of a masonry dam and storage reser-

* Presented at the Fall Meeting, Columbus, Ohio, October 12, 1927.
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voir on the river about 4.5 miles above the purification works. These improvements were fully described in a previous paper presented to the Society.*

In the fall of 1925 the city completed a second five-year program of important water-works improvements, this program including enlargement of the water purification and softening works and the construction of a second masonry dam and storage reservoir on the Scioto River.†

It is the purpose of this paper to describe the construction of the second masonry dam and reservoir, known as the O'Shaughnessy Dam and Reservoir.

Names of Dams

To avoid confusion, in case reference should be made to the paper and the report referred to, it will be well to state how the names, by which the two masonry dams are now known, came to be given. In the report recommending the construction of the second masonry dam, it was referred to, for the purpose of description, as the Dublin Dam, the proposed location being about 2 miles north of the Village of Dublin.

When the second five-year program was begun in 1920 Mr. Jerry O'Shaughnessy was the Superintendent of the Columbus Water-Works. Mr. O'Shaughnessy had been in the employ of the Water-Works Department for nearly fifty years, rising from the humblest position to the highest, and it was one of his greatest ambitions that the second five-year program should be carried to completion while he was still Superintendent of the Department, the work having been organized under a separate bureau of the Water-Works Department. This was not to be. Mr. O'Shaughnessy died on January 28, 1921, more than a year before the second dam was put under contract. The City Officials wished to recognize Mr. O'Shaughnessy's long years of service to the City of Columbus and decided to name the new dam for him. A bronze tablet, setting forth the action of the City Council in naming the new dam for Mr. O'Shaughnessy, has been mounted on the dam and bears the following wording:

O'Shaughnessy Dam

> By Resolution of the Council of the City of Columbus, Adopted, February 7, 1921, This Dam Was Named and Designated the "O'Shaughnessy Dam" in Honor of the Late Jerry O'Shaughnessy, Who so Faithfully and Well Served the City as Superintendent of Its Water Works for Many Years.

In the paper describing the construction of the works built under the first five-year program, beginning in 1904, the first masonry dam was referred to as the Scioto River Storage Dam and it was built while the late Julian Griggs, M. Am. Soc. C. E., was Chief Engineer of the Board of Public Service, the work under this program having been organized as practically a separate

* "The Improved Water and Sewage Works of Columbus, Ohio," by John H. Gregory, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXVII (1910), p. 206.

† This second five-year program was carried out substantially as recommended in a "Report on Enlargement of the Water Supply, Purification and Pumping Works of the City of Columbus, Ohio," by John H. Gregory, dated August 2, 1919.

branch of the Engineering Department of the Board. It was largely through the untiring efforts of Mr. Griggs that the first five-year program of improvements was started, and the first masonry dam was completed while he was still Chief Engineer of the Board. Mr. Griggs died on December 4, 1922. Having honored Mr. O'Shaughnessy for his long service to the City by naming the second dam for him, the City Officials then wished to honor Mr. Griggs for his very valuable services in connection with the construction of the first dam. Hence, the first dam was renamed the Julian Griggs Dam. The following is from the resolution adopted by the City Council on December 18, 1922:

"That the present storage dam in the Scioto River be and is hereby designated as 'The Julian Griggs Dam' in honor of Julian Griggs, who so faithfully and well served the City as Chief Engineer and who was one of the prime movers in the movement to construct said dam, and who had general supervision of its construction."

The two dams and reservoirs will be referred to, therefore, as the O'Shaughnessy Dam and O'Shaughnessy Reservoir and the Julian Griggs Dam and Julian Griggs Reservoir.

Capacity of Works

In the report previously referred to it was recommended that, in addition to increasing the size of the water purification and softening works and pumping station, the water supply works be enlarged to provide for an average water consumption of 55 000 000 gal. daily. This recommendation has been carried out and the O'Shaughnessy Reservoir, together with the Julian Griggs Reservoir, provides sufficient storage to meet this requirement. With an estimated water consumption of 110 gal. per capita daily the source of supply will now provide for a population of 500 000, estimated to be reached somewhere between 1950 and 1955.

In the operation of the purification and softening works and the pumping station, water is used for washing filters, flushing mixing chambers and settling basins, boilers, etc., and a study of the past operation of the Columbus works indicated that an allowance of 5%, or 2 750 000 gal. daily, should be made for water so used. Consequently, the water supply works will provide for an average raw-water draft of 57 750 000 gal. daily, corresponding to an average water consumption of 55 000 000 gal. daily.

In the carrying out of the second five-year program the water purification and softening works were enlarged, and, although it is not the intention in this paper to describe the enlargement of these works, their capacity before and after enlargement will be given, as a matter of reference. As first built, the purification works included ten rapid sand filters, each with a then rated capacity of 3 000 000 gal. daily, or a total of 30 000 000 gal. per day. While the piping and the filter rate controllers will permit running the filters at a somewhat higher output for a short period, in case of emergency, experience gained in the operation of the works for the first ten years demonstrated that it is not safe to run the filters, under Columbus conditions, at a rate greater

than 30 000 000 gal. daily for any considerable time. The maximum capacity of the first filter installation, therefore, must be put at 30 000 000 gal. daily.

The average daily capacity of a water purification works to supply filtered water is dependent on the maximum daily filter capacity, that is, the maximum safe rate at which it is possible to filter water, on the volume of filtered water stored in the filtered water reservoir and available for equalizing the fluctuating rates of water consumption, and on maximum and long continued high rates of water consumption. No general rule can be stated which will apply to all purification works, as the conditions at no two works are alike. From a study of Columbus conditions it was found that, with a maximum filter capacity of 30 000 000 gal. per day, an average daily capacity of only 22 300 000 gal. could be relied on.*

In the second five-year program the purification works were enlarged to a maximum filter capacity of 54 000 000 gal. daily by the addition of eight 3 000 000-gal. daily filters. The existing 10 000 000-gal. filtered water reservoirs were enlarged so as to have a total filtered water capacity of about 16 500 000 gal. Under these new conditions, following the method referred to, the purification works now have an average capacity of 40 000 000 gal. daily. A third and final enlargement of the purification works to an average daily capacity of 55 000 000 gal. will bring them up to the capacity of the water supply works.

In 1926 the City of Columbus had an estimated population of about 285 000. The water consumption for the same year was 27 400 000 gal. daily, including water supplied to a population of about 15 200 outside the city limits.

Water-Shed and Flow of the Scioto River

At the Julian Griggs Dam the Scioto River has a water-shed of 1 052† sq. miles, and at the O'Shaughnessy Dam, 988 sq. miles. On the water-shed the rock is of a limestone formation and is exposed at many places. Where the rock is covered, the drift is of varying thickness. The water is hard and is unsatisfactory for domestic and industrial uses unless softened. The average total hardness of the water for the 18 years, 1909-26 inclusive, was 272 parts per million, 160 parts being carbonate and the remaining 112 parts non-carbonate hardness. The maximum total hardness recorded on any one day was 495 parts per million. The average turbidity was 75 parts per million, but, during freshets and flood flows, turbidities in excess of 2 000 parts per million have been recorded. At such times, however, the total hardness drops to less than 100 parts per million. The maximum recorded turbidity was 2 250 parts per million. The water is softened and filtered before delivery into the distribution system.

* The method by which the average daily capacity was arrived at is a very interesting one and was described in detail, with graphs, by Mr. Gregory, in his report of August 2, 1919, on the enlargement of the water supply, purification, and pumping works of the City of Columbus. So far as the writers are aware, that was the first time this method was used. A description of the method may also be found in Chapter XIV, "Relation Between Filtered Water Storage and Filter Capacity," by John H. Gregory and Robert B. Morse, Members, Am. Soc. C. E., in the Manual of the American Water Works Association (1925), pp. 330–345.

† In *Transactions*, Am. Soc. C. E., Vol. LXVII (1910), p. 218, the area was given as 1 032 sq. miles. The new area of 1 052 sq. miles is from measurements made on the more recently issued topographical maps of the U. S. Geological Survey.

The daily flow of the Scioto River at the Julian Griggs Dam varies between very wide limits, from a minimum of 20 to 30 cu. ft. per sec. (roughly, 13 000 000 to 20 000 000 gal. daily), during the dry months of the summer

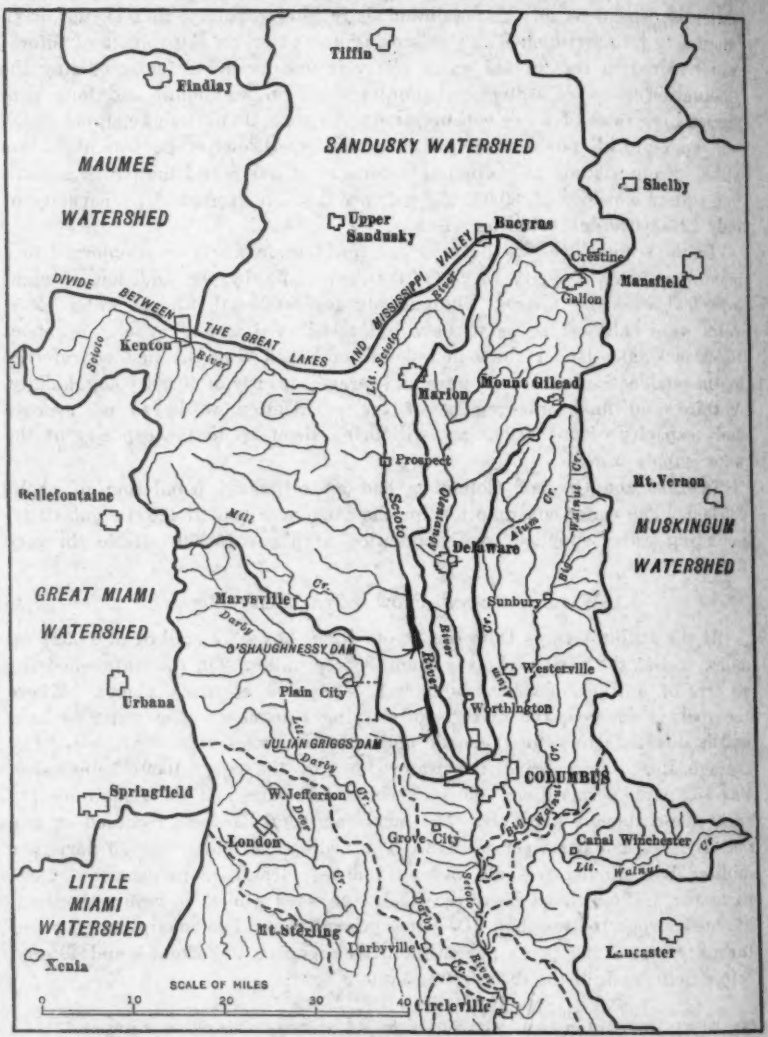

FIG. 1.—MAP SHOWING WATER-SHED OF SCIOTO RIVER, NEAR COLUMBUS, OHIO.

and early fall to upward of 1 000 cu. ft. per sec. during the winter and early spring months. Storage, therefore, is necessary in order to meet a water consumption demand of 55 000 000 gal. daily.

Beginning in February, 1909, records have been kept daily of the water level in the Julian Griggs Reservoir and of the opening of the sluice-gates in the dam when water was drawn from storage. From these records the daily flow was computed. Although there are a few gaps in the record, they are only of minor importance. The Julian Griggs Dam is a curved masonry structure of the overflow type, the length of the spillway being 500 ft., and the crest 30 ft. above low water in the river. From the dam the water flows down the river a distance of 4.5 miles to the intake of the purification works.

The water-shed of the Scioto River near Columbus and the location of the O'Shaughnessy and Julian Griggs Dams are shown on Fig. 1.

Storage Capacity Needed

From a study of the daily flow records of the Scioto River it was found that the maximum available storage needed to sustain a raw-water draft of 57 750 000 gal. daily at the Julian Griggs Dam, the lower of the two dams, would be in the last half of a year having daily flows similar to those which occurred in 1914. This storage may be readily obtained from the mass diagram shown on Fig. 2.

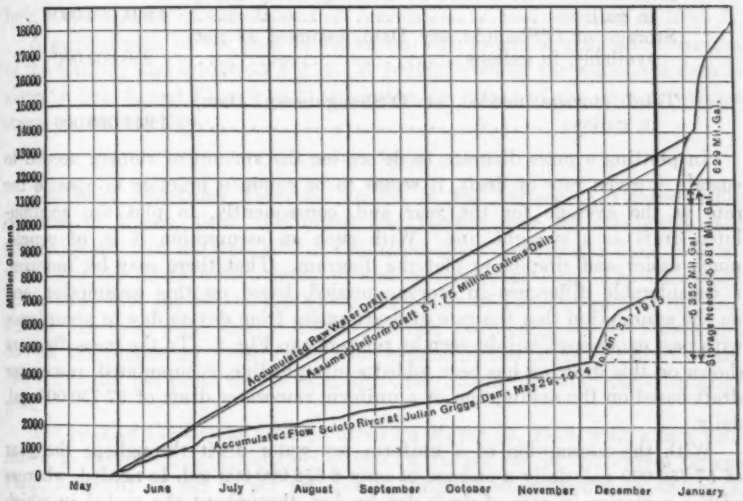

Fig. 2.—Storage Mass Diagram, Scioto River.

The lower curve gives the accumulated flow of the Scioto River at the Julian Griggs Dam from May 29, 1914, to January 31, 1915. The upper curve gives the accumulated raw-water draft for a similar time of the year, with an average raw-water draft for the year of 57 750 000 gal. daily, corresponding to an average water consumption for the year of 55 000 000 gal. daily. In computing the data for the plotting of this curve, it was assumed that the ratio of the raw-water draft, day by day, to the average daily raw-water

draft for the year would be the same as the ratios experienced on corresponding days in the period June, 1913, to January, 1914, it having been found, from a study of the daily water consumption records, that the maximum daily rates of long continued water consumption occurred in the period mentioned.

From these two curves it is seen that a total available storage of 6 981 000 000 gal. is needed above the Julian Griggs Dam to sustain a raw-water draft of 57 750 000 gal. daily. The available storage needed above the O'Shaughnessy Dam was then found, as follows:

Total available storage needed, in gallons.....		6 981 000 000
Available storage at Julian Griggs Dam, without flash-boards, in gallons	1 487 000 000	
Additional storage at Julian Griggs Dam, with 4-ft. flash-boards, in gallons	500 000 000	
Total available storage at Julian Griggs Dam, with 4-ft. flash-boards, in gallons	1 987 000 000	1 987 000 000
Available storage needed at O'Shaughnessy Dam, in gallons		4 994 000 000
Storage at O'Shaughnessy Dam, assumed as not available, in gallons		250 000 000
Total storage needed at O'Shaughnessy Dam, in gallons		5 244 000 000

In plotting a mass diagram to determine the amount of storage needed to sustain a given rate of draft, it seems to be common practice to assume the rate as the average for the year and, consequently, to plot the accumulated draft as a straight line. With such an assumption it is, of course, much easier and simpler to plot the diagram. That there may be, however, a considerable difference in storage needed, based on this assumption and on the assumption that the rate of draft varies from day to day in accordance with past experience, will be seen by reference to Fig. 2. To the mass diagram shown on this drawing has been added a straight-line, accumulated, raw-water draft based on the assumption of a uniform raw-water draft of 57 750 000 gal. daily.

With the assumption of a uniform raw-water draft throughout the year of 57 750 000 gal. daily a storage of only 6 352 000 000 gal. is needed, whereas with a variation in rate of draft day by day, throughout the period in which storage is needed, in accordance with past experience during a similar period of the year, a storage of 6 981 000 000 gal. is seen to be necessary, an increase of 629 000 000 gal.; that is, 9.9% more storage is needed in order to maintain higher rates of daily raw-water draft which are to be expected during the period when storage is necessary than under the assumption of a uniform daily rate of raw-water draft throughout the year.

From discussion with other engineers, as well as from examination of reports and books in which mass diagrams are given, it has appeared to

the writers that many have not appreciated that the assumption of a uniform rate of draft throughout the year may give too small a storage, about 10% in the case just illustrated. Therefore, they have taken this opportunity to bring the matter to the attention of engineers.

Type of Structure

The O'Shaughnessy Dam is a concrete masonry dam, of the gravity overflow type, straight in plan, and 1 750 ft. in length, including the approaches. The masonry portion of the dam is 879 ft. long, measured face to face of copings on the abutments. The two approaches have a total length of 871 ft., but they are not of equal length, the east approach being slightly longer than the west one.

Crossing the dam, above the overflow or spillway section, is a reinforced concrete arch bridge of twelve spans, each span measuring 64 ft. 6 in. in the clear. The net length of the spillway, under the bridge, after deducting the length of the bridge piers and the outlet gate-house, is 765.7 ft.

The crest of the spillway is 64 ft. above low water in the river on the down-stream side of the dam and 84 ft. above the rock foundation. The roadway on the bridge is 21.3 ft. above the crest of the spillway making, therefore, a total height of dam from rock foundation to roadway level of 105.3 ft.

A general plan of the dam is shown on Fig. 3, and a plan and elevation of the spillway and bridge on Fig. 4. A general view of the completed dam is given in Fig. 5, and Figs. 6 to 11, inclusive, are views of the dam at various stages of construction.

Capacity of Spillway

The most disastrous floods on record in the history of Central Ohio occurred in March, 1913. Columbus and Dayton were the two cities that suffered most. On the west side of Columbus great damage was done to property when the levees were overtopped and 93 lives were lost. The Olentangy River flows into the Scioto just above the center of the city and, at the confluence, the total water-shed is 1 612 sq. miles, 1 076 being on the Scioto and 536 on the Olentangy. Fortunately for Columbus the peak-flood flows of the two rivers did not occur at the same time, the Scioto following the Olentangy peak.

The peak of the Scioto flood occurred on March 25, 1913, with a flow estimated to have been approximately 80 000 cu. ft. per sec. at the Julian Griggs Dam, or about 76 cu. ft. per sec. per sq. mile of water-shed. At the peak, the depth of water passing over the 500-ft. spillway of this dam was 12.8 ft.

Assuming the same flood flow per square mile at the site of the O'Shaughnessy Dam, the discharge from the 988 sq. miles of water-shed above this point would have been about 75 000 cu. ft. per sec. After mature consideration of the subject it was finally decided to provide spillway capacity at the new dam for a flood of 110 000 cu. ft. per sec., or about 50% in excess of the 1913 flood, and also to increase the length of the spillway about 50% over that of the Julian Griggs Dam. Under these assumptions to discharge 75 000 cu. ft. per sec. over the spillway of the O'Shaughnessy Dam will

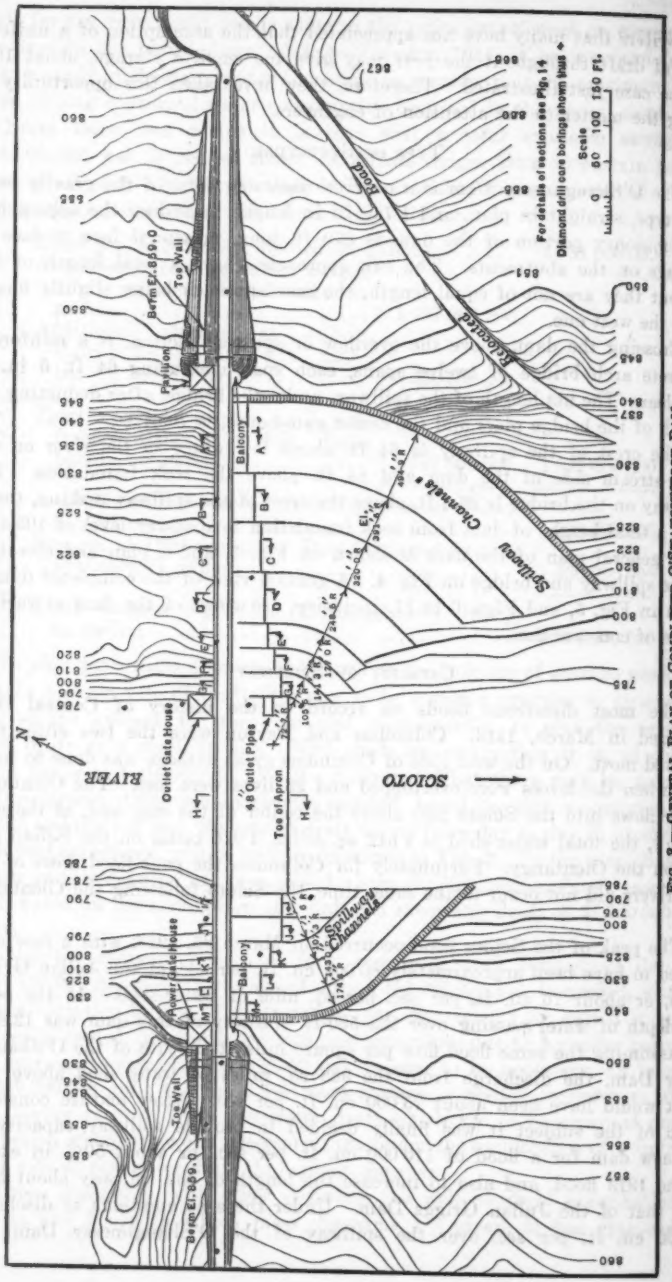

FIG. 8.—GENERAL PLAN OF COMPLETED O'SHAUGHNESSY DAM.

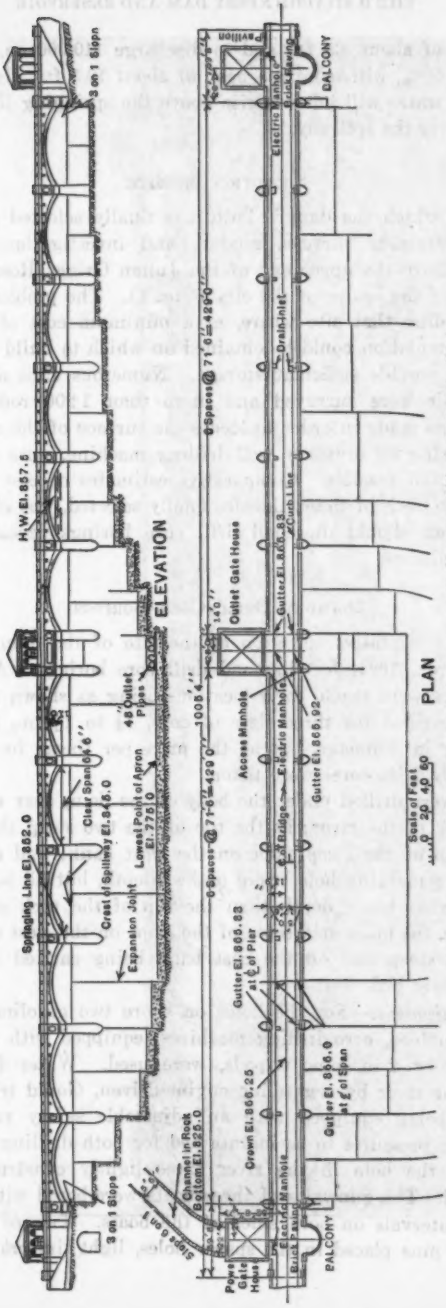

FIG. 4.—PLAN AND ELEVATION OF SPILLWAY AND BRIDGE, O'SHAUGHNESSY DAM.

require a head of about 9.5 ft., and to discharge 110 000 cu. ft. per sec., the 1913 flood plus 50%, will require a head of about 12.5 ft. Under the extreme flood condition water will have to rise above the springing line of the arches of the bridge over the spillway.

Selection of Site

The site on which the dam is built was finally selected after more than a year of preliminary surveys, studies, and investigations. This site is about 4 miles above the upper end of the Julian Griggs Reservoir and about 16 miles north of the center of the city (Fig. 1). The problem resolved itself into one of finding that site where, at a minimum cost of construction, a suitable rock foundation could be obtained on which to build the dam and, at the same time, provide sufficient storage. Numerous sites above the Julian Griggs Reservoir were surveyed and more than 1 200 rod soundings and spud-drill borings made in order to locate the surface of the rock. The spud-drill borings, using an ordinary well-drilling machine, were made where rod soundings were not feasible. Comparative estimates of cost were then compiled and, on the basis of these, the site finally selected, was chosen, subject to revision, however, should diamond drill core borings reveal unsatisfactory sub-surface conditions.

Diamond Drill Core Borings

Following the tentative selection of the site of the dam a contract was awarded in March, 1921, for diamond drill core borings. All told, twenty-two such borings were made, their locations being as shown on Fig. 3. The specifications provided for three sizes of core, 1¼ to 1½ in., 1½ to 2 in., and 2 in. or greater in diameter, but as the price per linear foot was the same for all sizes, only 2-in. cores were taken.

Deep holes were drilled under the body of the dam, near the water's edge, on the east bank of the river; at the top of the two steep slopes on the east bank; at the top of the steep slope on the west bank; and at the center of the river. The remaining holes were of less depth, but all borings were carried to a somewhat lower depth than the top of the rock at the next hole toward the river, the holes at the top of the slope on the west bank, and at the top of the first steep rise on the east bank being carried down below the bottom of the river bed.

Drilling Equipment.—For the holes on shore two gasoline engine-driven, Sullivan friction feed, core-drilling machines, equipped with single core barrels and served by 2-in. pipe tripods, were used. Water for drilling was pumped from the river by a gasoline engine-driven, Gould triplex pump, the discharge pipe being equipped with an adjustable safety valve which permitted very even pressures to be maintained for both drilling and testing.

For drilling the hole in the river three lightly constructed high-sided dories were used. The gunwales of these boats were bored with pairs of holes at about 3-ft. intervals on both sides of the boats. Two of the boats were lashed together, pins placed in the spacer holes, light timbers placed between

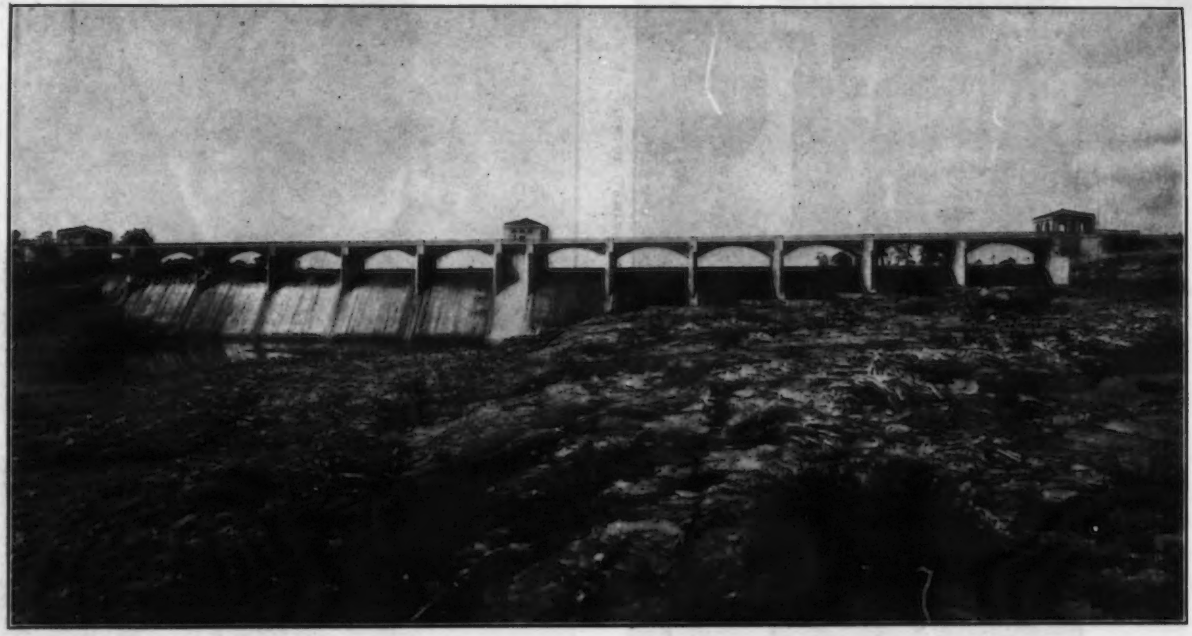

FIG. 5.—VIEW OF COMPLETED O'SHAUGHNESSY DAM.

FIG. 6.—VIEW OF O'SHAUGHNESSY DAM, LOOKING EAST, JUNE 21, 1923.

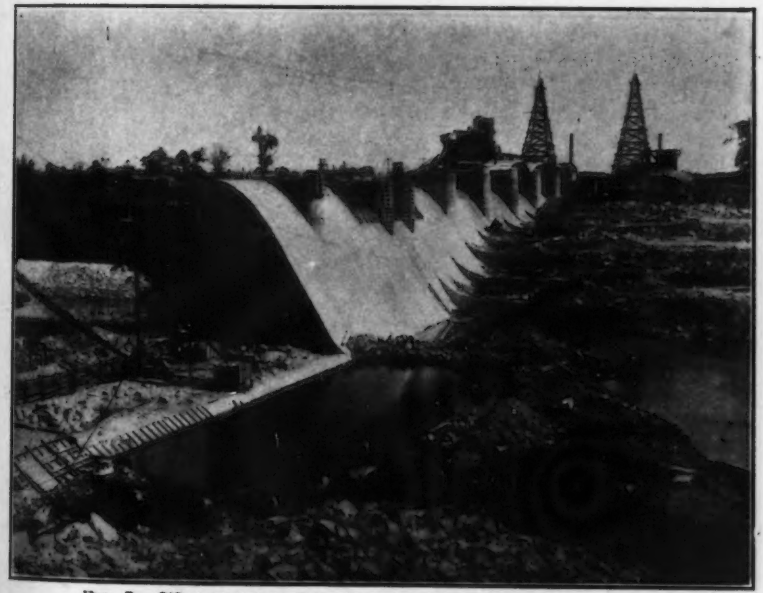

FIG. 7.—O'SHAUGHNESSY DAM, LOOKING EAST, SEPTEMBER 27, 1923.

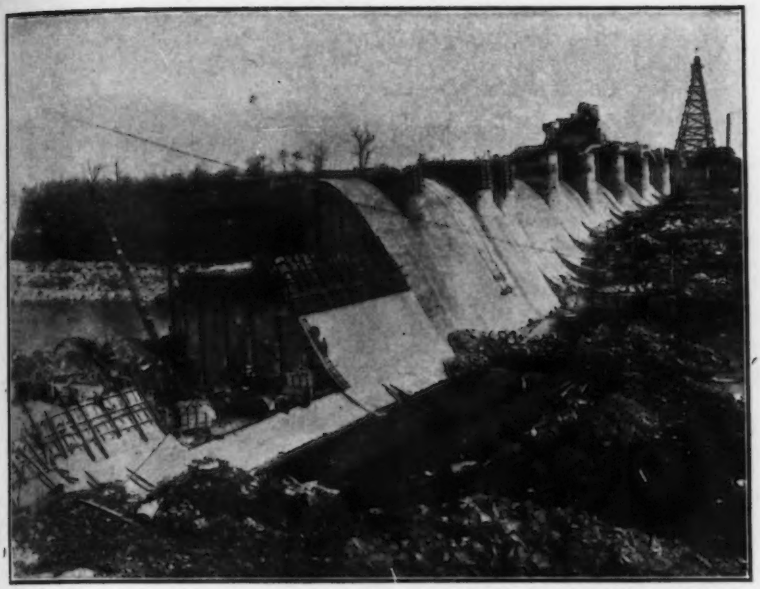

FIG. 8.—O'SHAUGHNESSY DAM, LOOKING EAST, OCTOBER 25, 1923.

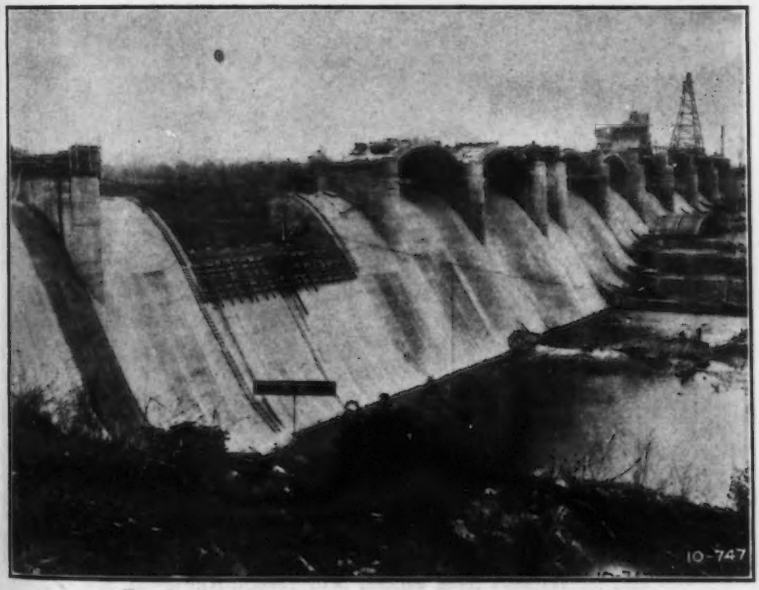

FIG. 9.—O'SHAUGHNESSY DAM, LOOKING EAST, MAY 9, 1924.

FIG. 10.—O'SHAUGHNESSY DAM, LOOKING EAST, JUNE 13, 1924.

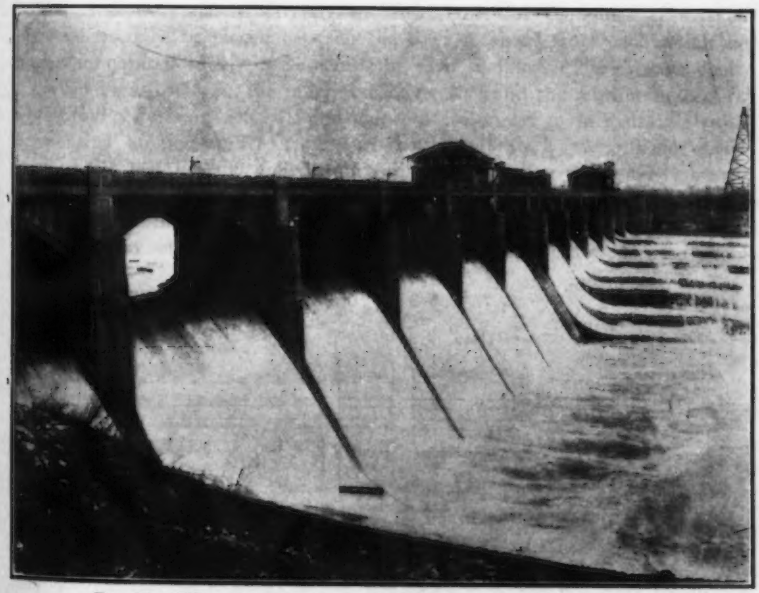

FIG. 11.—O'SHAUGHNESSY DAM, LOOKING EAST, FEBRUARY 13, 1925.

the pins and lashed down, and 2-in. plank laid on the timbers to form a floating platform. The drilling machine was skidded on to this platform from the bank, the whole being towed to position by the third boat and anchored with four heavy ship's anchors.

The equipment furnished also included a small portable shop and tools for making repairs and for resetting the diamonds in the cutting bits, this work all being done on the job. The equipment for testing is described subsequently.

Drilling.—Where the rock was not exposed a 4-in. casing was driven to the top of ledge rock by one of three methods: First, by driving the casing, pulling it up, and knocking out the earth core, and then repeating the operation; second, by jetting out the earth and driving the casing; or, third, where slabs of rock embedded in the earth were encountered, by drilling with a spud bit. All casings were left in place and capped. In exposed rock, short pieces of casing were cemented into the holes in order to save them for grouting during construction.

The drilling of the holes was done in the usual manner, a complete log of each hole being kept, the notes reporting any changes in the color or quantity of the discharged water containing the cuttings, the depth to the bottom of the cutting bit at each pull, a sounded depth, and the length of core obtained at each pull, as well as notes on the physical characteristics of the rock, etc.

The cores were placed directly in boxes built for the purpose, marked blocks of wood being placed between the rock obtained from each pull; the boxes were covered with wire screening as soon as filled. The average depth of the holes was 34.55 ft., the maximum being 65.0 and the minimum, 19.3 ft.

Hydraulic Tests.—Each hole was subjected to one or more hydraulic tests, water being applied under a pressure equivalent to a static head corresponding to Elevation 860, for the purpose of determining the water-tightness of the rock and also for locating any open seams or fissures. More and better information on these essential points was obtained from the hydraulic tests than from any other source, not excluding the information secured from the cores.

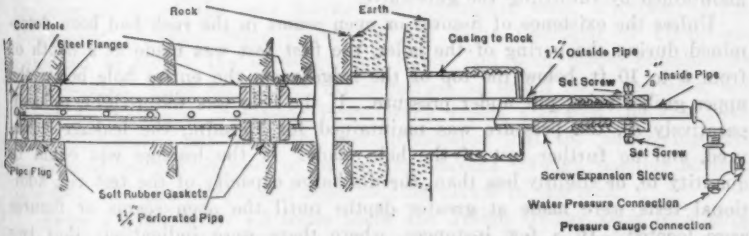

FIG. 12.—HYDRAULIC TEST APPARATUS FOR DIAMOND DRILL CORE BORINGS.

Water, under the pressure noted, was applied to the test hole through a double string of pipe of lengths necessary to reach the point to be tested. The outside pipe, shown in Fig. 12, was 1¼ in. in diameter and was made up

with a set of soft rubber gaskets at the bottom, a short length of pipe above, with several holes drilled through the sides of the pipe, a second set of gaskets, and a second piece of pipe reaching to above the ground. The gaskets were held between iron flanges, the gaskets and flanges having an outside diameter slightly smaller than that of the test hole. The inside pipe, ¾ in. in diameter and about 1 ft. longer than the outside string of pipe, passed through the upper set of flanges and gaskets, through the upper flange and gaskets of the lower set, and was screwed into and through the bottom flange. This inside pipe was closed at the lower end with a removable plug, was perforated with several holes just above the lower set of gaskets, and was fitted with an expanding screw sleeve at the top. The upper half of the sleeve could be securely fastened to the inside pipe with set screws, and the lower half bore directly against the top of the outside pipe. By screwing the sleeve out, the gaskets could be compressed until water-tight connections were made with the sides of the hole and with the outside of the inside pipe. Pressure could be applied from the inside pipe, through the holes in both pipes, to the rock sides of the test hole between the top and bottom gaskets, any desired length of hole being put under pressure by simply placing the desired length of perforated outside pipe between the top and bottom gaskets; or, the entire portion of the hole below the upper gasket could be put under pressure by removing the plug at the bottom of the inside pipe.

Water for testing was supplied under pressure through a short length of hose, the quantity and pressure being controlled by a gate-valve, and the quantity measured by a ¾-in. service meter. The pressure was shown on a small pressure gauge located close to the connection to the top of the ¾-in. inside pipe. Due to the small size of the inside pipe and the high friction loss, it was necessary to determine, in advance of testing the holes, the maximum quantity of water which various lengths of rig would discharge, taking into account the range of pressure required by the elevation of the ground at the several holes. Determinations, sufficient in number to cover the various lengths of test piping and pressures required, were made of the discharge capacity of the pipe by laying the pipe on the ground and measuring the discharge into the air, a constant pressure at the gauge being maintained by throttling the gate-valve.

Unless the existence of fissures or open seams in the rock had been determined during the boring of the holes, the first test was made at a depth of from 5 to 10 ft. below the top of the ledge rock, the entire hole below the upper gasket being put under pressure. If the hole was water-tight, or comparatively so, the pressure was maintained for 15 min., the leakage measured, and no further test of the hole made. If the leakage was equal in quantity to, or slightly less than, the discharge capacity of the test rig, additional tests were made at greater depths until the open seams or fissures were located. In a few instances, where there were indications that two or more small seams had been encountered, the pressure was confined to the space between the gaskets, with the gaskets set so as to be above and below one of the seams, the leakage into this seam measured, and then similar tests were made on the other seams.

Where seams were at a considerable depth, and it was desirable to determine the condition of the rock above the seam, a permanent seal was made by filling the hole with cement mortar to a point above such seam. After a sufficient time had elapsed to permit the mortar to set, the hole above the seal was tested in the usual manner.

Percentage of Core.—The length of core recovered, as measured in position in the core boxes, was 84% of the total depth of all the holes measured from the top of the solid rock to the bottom of the holes.

Cost.—The prices paid for making the diamond drill core borings, for temporary and permanent seals, for conducting hydraulic tests, and for casing left in place were, as follows:

Cores, per linear foot	$4.50
Temporary seals, each	15.00
Permanent seals, each	15.00
Hydraulic tests, each	15.00
Casing, left in place, per pound	0.06

In paying for the core borings the linear-foot measurement was made from the surface of the ground to the bottom of the hole. A temporary seal was paid for each time the testing rig was set in a hole, the pressure turned on, and the flow measured. If the pressure was maintained and the flow measured for a period of 15 min. then payment for a hydraulic test was made, in addition to payment for the temporary seal. The total cost of the diamond drill core-boring investigation was $5 068, exclusive of engineering.

Geology of the Site

The diamond drill core borings and hydraulic tests made at the proposed site indicated that there were no sub-surface conditions seriously detrimental to its selection. The sub-surface investigation showed, however, that grouting of the underlying rock would be required at certain points and that further extension of the grouted areas would be desirable, but as a measure of insurance rather than of necessity.

Before passing final decision on the selection of the site, and following the completion of the core borings and hydraulic tests, Professor J. A. Bownocker, of Ohio State University and State Geologist of Ohio, was requested to review the investigations which had been made, examine the cores secured, make such other investigations as might seem desirable, and report his findings. Professor Bownocker's report approved the site without qualification, but made the recommendation that the underlying rock be grouted for the purpose of preventing leakage through possible solution channels, cleavage planes, and open bedding planes in the rock.

From a point about 4.5 miles below the Julian Griggs Dam to a point well above the limits of back-water from the O'Shaughnessy Dam, the Scioto River flows through a valley cut in the Devonian limestone. The lower reaches of this course cut through the Columbus limestone at normal water level, with the Delaware limestone outcropping in the higher lying portions of the

valley adjacent to the river. Near the Village of Bellepoint, about 6.5 miles above the site, the next lower formation appears in the bed of the river, namely, the Monroe limestone, the part here represented being of the Silurian Age.

Overlying the limestone, at all points in the valley where slopes are flat enough to prevent erosion, is a mantle of Wisconsin glacial drift, consisting very largely of yellow boulder clay with a thin top layer of clay loam, the boulder clay being an ideal material for embankments and the loam suitable for top dressing.

Near Columbus the Columbus limestone has an average thickness of about 105 ft. The Delaware limestone lies above the Columbus limestone, the contact being characterized by the bone-bed, notable among geologists for its high content of fossilized plates, teeth, and other bony structure of Devonian fish life. Below the Columbus limestone is the Monroe limestone, the two being separated, however, by an unconformity. The basal layer of the Columbus formation is commonly a conglomerate which consists of pebbles of Monroe limestone embedded in a matrix of smaller Columbus limestone and quartz sand.

The Columbus formation at the site consists of this basal layer, surmounted by many layers of limestone of varying thickness ranging from 2 or 3 in. to 4 and 5 ft. and varying in chemical composition from a true dolomite to an almost pure calcite, with physical characteristics almost as variable as the chemical composition.

Excavation for that part of the dam west of the outlet gate-house and in the easterly half of the river bed, developed the fact that the bed of the river, at the site, was in the massive lower layers of the Columbus formation and that excavation to the depth shown on the contract drawings would require the removal of much of this massive stone, a procedure which was unnecessary and which was immediately changed by decreasing the depth of excavation, where the rock foundation warranted, to a minimum of about 5 ft. under the dam proper, and with a somewhat less depth under the apron.

Excavation for the cut-off trench in the same location penetrated to a point about 1 ft. below the bottom of the basal conglomerate. The conglomerate was found to be from 6 to 8 in. in thickness and to have both above and below it, and throughout most of its extent, a thin layer of plastic blue clay. This discovery, which was not developed by the borings, led to the decision to carry all the cut-off trench excavation, across the river bed, to a point at least 1 ft. below the bottom of the basal conglomerate.

The dip of the rock in the Columbus District is from 20 to 30 ft. per mile to the east, or a little south of east. At the site it was found to approximate the larger figure and the cut-off trench was made to follow the variation in the level of the rock, being a little higher at the west end of the river section than at the easterly end under the outlet gate-house.

Excavation for the core wall at both ends of the dam encountered a small depth of the basal layers of the Delaware limestone underlaid by the

bone-bed and the top of the Columbus formation. The total thickness of the Columbus formation was about 87 ft., being less than the usual thickness for the Columbus District.

None of the excavated rock was too soft for use as coarse aggregate for concrete and the thicker layers made excellent stone for slope paving. It was necessary, however, to exclude the use of the softer stone for the wearing surface of the macadam roads constructed adjacent to the dam.

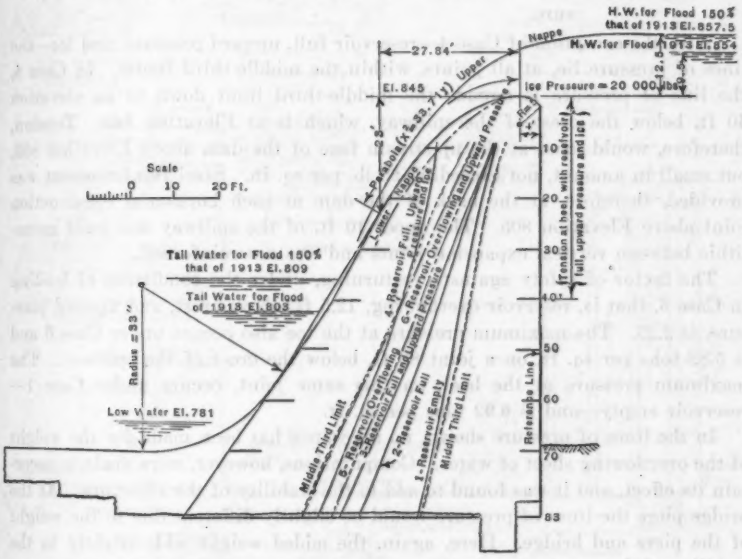

FIG. 13.—SPILLWAY SECTION, SHOWING LINES OF PRESSURE, O'SHAUGHNESSY DAM.

DESIGN OF SPILLWAY SECTION

The maximum section of the spillway, as constructed in the clear between the piers supporting the bridge above, is shown on Fig. 13. This section also shows the lines of pressure under various conditions of loading. In the design of the section the following assumptions were made:

Weight of masonry, in pounds per cubic foot..... 145
Weight of water, in pounds per cubic foot..... 62.5
Ice thrust, acting at crest, in pounds per linear foot.....20 000
Upward pressure: Full head pressure at heel decreasing uniformly to 0 at the toe, acting over two-thirds of the area.
Maximum head over spillway, 12.5 ft., for flood 150% that of 1913.
Surface of tail-water, 36 ft. below crest, for flood 150% that of 1913.
No silt in reservoir.
No wind when reservoir is empty.

The lines of pressure shown are for six different conditions of loading as follows:

Case 1.—Reservoir empty.
Case 2.—Reservoir full.
Case 3.—Reservoir full and upward pressure.
Case 4.—Reservoir full, upward pressure, and ice.
Case 5.—Reservoir overflowing, 12.5 ft. above crest.
Case 6.—Reservoir overflowing, 12.5 ft. above crest, and upward pressure.

With the exception of Case 4—reservoir full, upward pressure, and ice—the lines of pressure lie, at all points, within the middle-third limits. In Case 4, the line of pressure is outside the middle-third limit down to an elevation 40 ft. below the crest of the spillway, which is at Elevation 845. Tension, therefore, would exist at the up-stream face of the dam above Elevation 805, but small in amount, not exceeding 26 lb. per sq. in. Steel reinforcement was provided, therefore, at the back of the dam at each horizontal construction joint above Elevation 805. The upper 10 ft. of the spillway was built monolithic between vertical expansion joints and was not reinforced.

The factor of safety against overturning, under the conditions of loading in Case 6, that is, reservoir overflowing, 12.5 ft. above crest, and upward pressure, is 2.25. The maximum pressure at the toe also occurs under Case 6 and is 5.82 tons per sq. ft. on a joint 83 ft. below the crest of the spillway. The maximum pressure at the heel, on this same joint, occurs under Case 1—reservoir empty—and is 6.92 tons per sq. ft.

In the lines of pressure shown, no allowance has been made for the weight of the overflowing sheet of water. Computations, however, were made to ascertain its effect, and it was found to add to the stability of the structure. At the bridge piers the lines of pressure would be slightly different due to the weight of the piers and bridge. Here, again, the added weight adds slightly to the stability of the structure.

On the spillway section, Fig. 13, are shown the upper and lower nappes for an overflowing sheet of water having a depth of 12.5 ft. corresponding to the 1913 flood plus 50%, and it will be noted that the shape of the crest is such that the face of the masonry projects into the sheet of water well beyond the lower nappe. The maximum and minimum sections of the spillway, as well as the intermediate ones, are shown on Fig. 14.

Reinforced Concrete Arch Bridge Over Spillway

At Powell Road, about 2 000 ft. above the dam, a county highway crossed the river on a 2-span, steel, truss bridge, 240 ft. in length. With the filling of the reservoir this bridge would have been submerged. As it was not feasible to abandon a crossing of the reservoir or river at or near this point, the choice then lay between raising the existing bridge about 51 ft., adding new trusses at each end, and building long expensive approaches, and the construction of a new bridge. Comparative estimates showed that it would be considerably cheaper to build a bridge across the dam and to abandon the crossing at

Powell Road. This solution of the problem was adopted after it had been submitted to and approved by the County Commissioners of Delaware County, in which the dam and reservoir are located. It was also of advantage to the city to have a bridge across the dam, as ready access would thereby be provided from one side of the dam to the other and also to the two gate-houses.

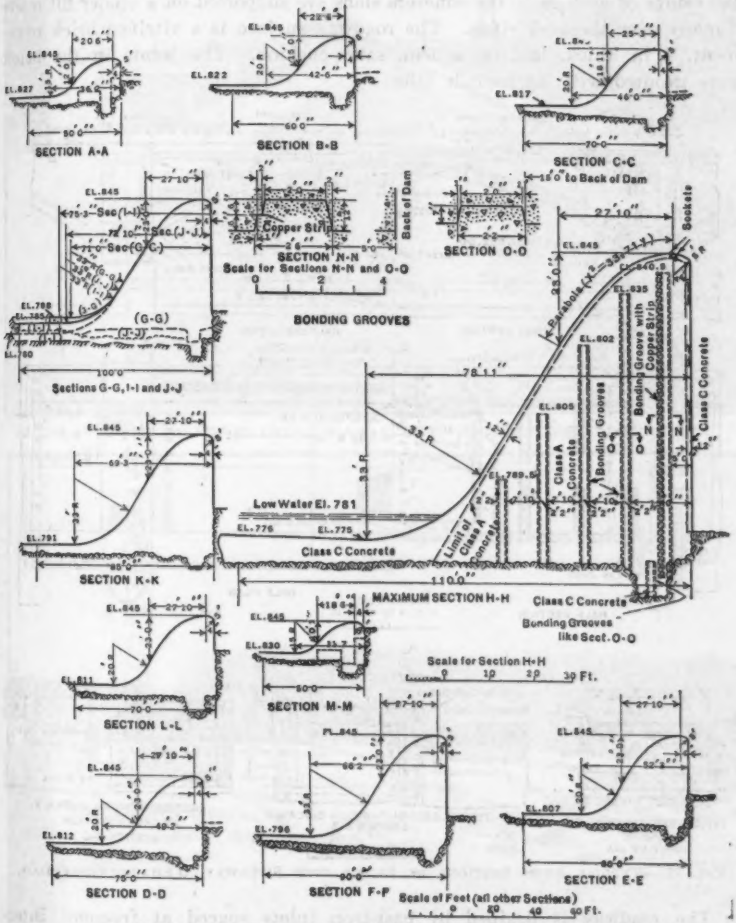

FIG. 14.—SECTIONS OF SPILLWAY, O'SHAUGHNESSY DAM.

The bridge consists of twelve reinforced concrete arches and has a total length of 879 ft., measured face to face of copings on the abutments at each end of the spillway. The arches are segmental, the intrados being struck on a radius of 52 ft. 9⅞ in. Each arch has a clear span of 64 ft. 6 in. and a rise of 11 ft. The thickness at the crown is 18 in., and at the springing line, 3 ft.

At right angles to the axis of the bridge, measured along the intrados, the arches have a length of 19 ft. 4 in.

The roadway over the arches has a clear width of 17 ft. 6 in., and is carried by reinforced concrete slabs supported by spandrel and curtain-walls extending down to the arch rings, except that, for a length of 21 ft. 6 in. at the center of each arch, the concrete slabs are supported on a cinder fill made directly over the arch rings. The roadway surface is a vitrified brick pavement, 4 in. thick, laid on a 1-in. sand cushion. The joints in the brick were grouted with an asphalt filler.

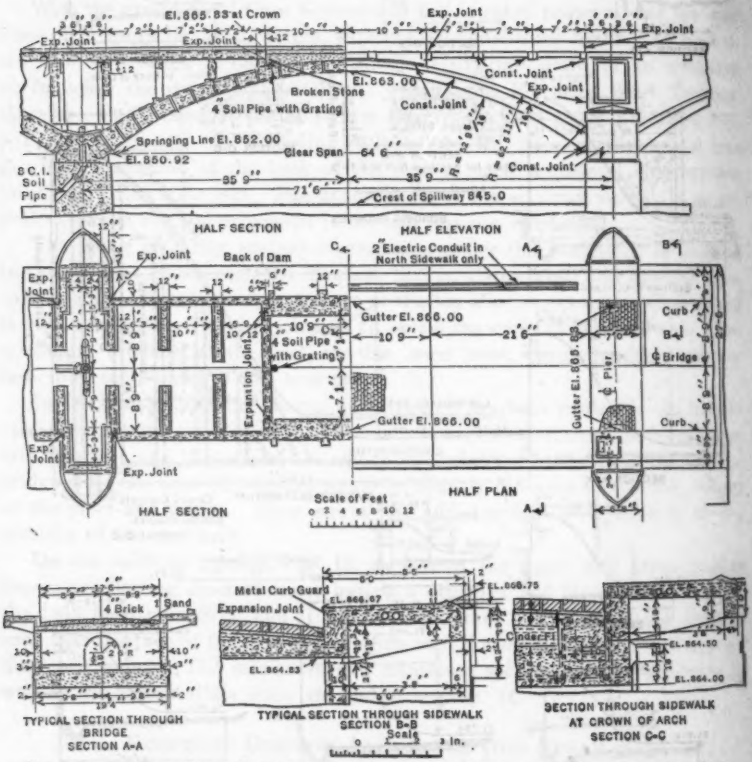

FIG. 15.—TYPICAL ARCH SECTIONS OF BRIDGE OVER SPILLWAY, O'SHAUGHNESSY DAM.

The roadway is drained by cast-iron inlets spaced at frequent intervals along each side of the pavement. The storm water is collected above the arches and is discharged through the arches, or through the piers, depending on the arrangement of the details.

On each side of the roadway is a 4-ft. sidewalk, consisting of a reinforced concrete slab carried on cantilever brackets anchored to the arch ring at the center of the arch and to the spandrel walls over the haunches

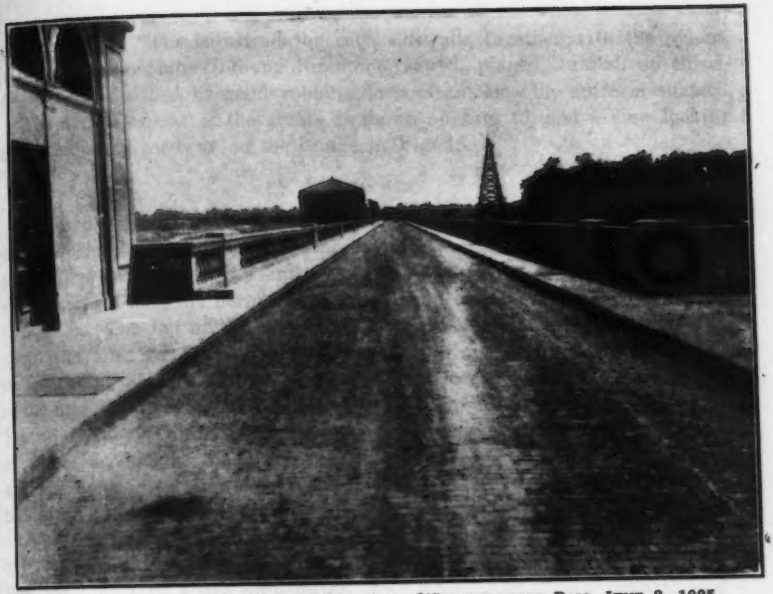

FIG. 16.—VIEW OF BRIDGE OVER SPILLWAY, O'SHAUGHNESSY DAM, JUNE 3, 1925.

FIG. 17.—O'SHAUGHNESSY DAM, EAST SPILLWAY CHANNELS, FEBRUARY 13, 1925.

at other points. The balustrade on each sidewalk, together with the pylons, is constructed of blue Indiana limestone, sawed, planed, turned, or chisel-dressed, and finished by sand rubbing to a clean, smooth, uniform surface.

The details of one of the arches is shown on Fig. 15, and a view looking along the paved roadway on the bridge in Fig. 16.

Approaches

At each end of the spillway section the approach consists of a rolled earth embankment or dike with concrete core wall, Fig. 18. The approaches are 25 ft. 6 in. wide on top and have side slopes of 1 on 2, except that the stone paving below the berm, on the up-stream side of the dike, has a slope of 1 on 3. The top of the dike is at Elevation 866.25, that is, 21 ft. 3 in. above the crest of the spillway and 8 ft. 9 in. above a high-water level, corresponding to the 1913 flood plus 50 per cent. On the reservoir side of the dike the 1 on 2 slope terminates at a berm 10 ft. in width and located 1 ft. 6 in. above high water. The slope paving begins at the outer edge of the berm, and, at the lower end, is either keyed into the rock or abuts against a concrete toe-wall. It is from 12 to 18 in. in thickness and is bedded on 12 in. of free-draining broken stone.

The concrete core wall is 3 ft. wide at the top, has a batter of ½ in. in 1 ft. on each side, and is keyed 12 in. into solid rock. The top is at Elevation 858.5, or 1 ft. above high water.

The roadway on each approach is of bituminous macadam, the width, 17 ft. 6 in., being the same as the clear width, between curbs, of the roadway on the bridge over the spillway. On each side of the roadway the edge of the embankment is protected by a fence built of reinforced concrete posts and rails. Just before the approaches connect with the masonry abutments at each end of the spillway, concrete steps on each side of the roadway provide access to the natural surface of the ground.

Outlet Gate-House

The outlet gate-house (Fig. 19) is 32 ft. square, inside dimensions, and is located at the center of the spillway, projecting out from the up-stream side of the dam. When water is drawn from storage it is taken through the gate-chamber immediately below. In the gate-chamber are two sets of chambers or wells, each set consisting of two separate wells connected by sluice-gates. There is no connection between the two sets of wells, thus providing duplicate means of drawing water from the reservoir. In case of repair to any sluice-gate one set of wells can be shut off.

The up-stream well connects with the reservoir through six 4-ft. by 7 ft. 6-in. rectangular openings, and immediately behind these openings are three sets of stop-plank grooves. In the up-stream groove are placed coarse bar screens, Fig. 20, having clear openings of 3⅝ in. Water drawn from the reservoir, is discharged into the river immediately below the dam and flows into the Julian Griggs Reservoir. A fine screen, therefore, is not needed at present, and the coarse screen serves simply to keep out floating débris and thus protect the sluice-gates. The two inner grooves are for a double set of stop-

planks, Fig. 20, which can be inserted should it become necessary to inspect or repair a sluice-gate. The stop-planks are stored in grooves provided for the purpose in two special wells. The stop-plank lifter is also shown in Fig. 20.

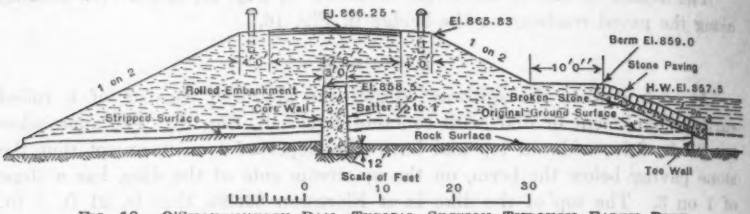

FIG. 18.—O'SHAUGHNESSY DAM, TYPICAL SECTION THROUGH EARTH DIKE.

Each up-stream well is connected with the corresponding down-stream well by three 36 by 60-in. rectangular openings, each opening being fitted with a rectangular sluice-gate of the same size as, and placed at the up-stream end of, the opening. These sluice-gates are set at different elevations so as to permit the drawing of water from storage at three different levels. From each down-stream well water is taken through two 30 by 60-in. sluice-gates into two 48-in., cast-iron pipes passing through the dam and terminating at the up-stream end of the apron. The ends are submerged so that the discharge is into the water cushion above the apron.

Each well is fitted with a ladder extending from the gate-house floor to the bottom of the well, and at various elevations pockets have been left opposite each other in the side walls of the well for the insertion of temporary platforms, so that access can be had to the sluice-gates and to the steady bearings on the sluice-gate stems.

At the bottom of the gate-chamber four 48-in., cast-iron pipes were placed, Fig. 19, below the bottom of the up-stream wells and connecting the down-stream wells with the up-stream face of the gate-chamber. These pipes took the ordinary flow of the river after the dam and gate-chamber had been constructed to an elevation above low water. After the completion of the dam each pipe was closed at the up-stream end by a cast-iron, blank flange, the pipe filled with concrete, and the lower end closed with another blank flange. Grout was then pumped into the pipe so as to fill it solid. In addition, the down-stream end of the pipe was permanently sealed, outside, by a concrete bulkhead grouted in.

The gate-house superstructure is built of cut and dressed blue Indiana limestone, matching that in the balustrade on the bridge, and is lined inside with a light-colored, salt-glazed face brick. The roof is constructed of steel trusses carrying concrete slabs 3 in. in thickness. The covering is of bright red mission tile laid on a bedding of Nalecode nailing concrete, 1½ in. thick. The doors and windows are of hollow metal construction built of heavy hard copper, stiffened, and reinforced. The windows are glazed with ¼-in. polished wire woven glass. The superstructure houses the hand-operated, ball-bearing, sluice-gate stands and also a 2-ton, hand-operated crane for raising and lowering the screens, stop-planks, etc.

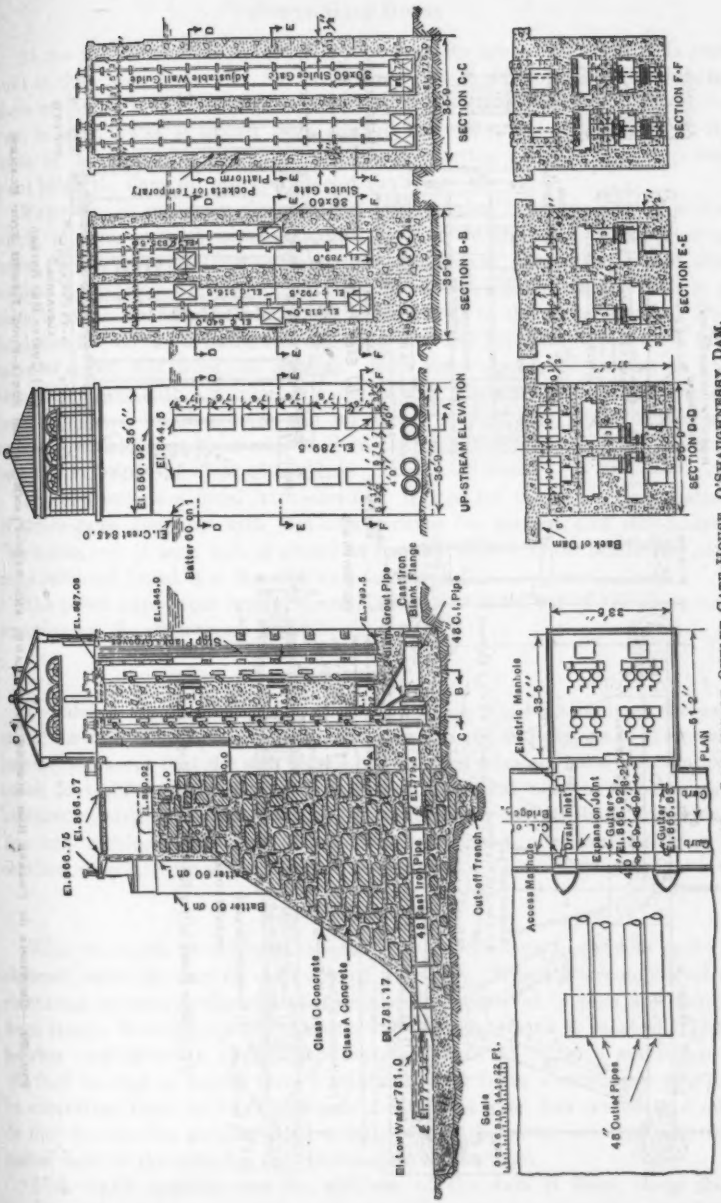

FIG. 19.—SECTIONS OF OUTLET GATE-HOUSE, O'SHAUGHNESSY DAM.

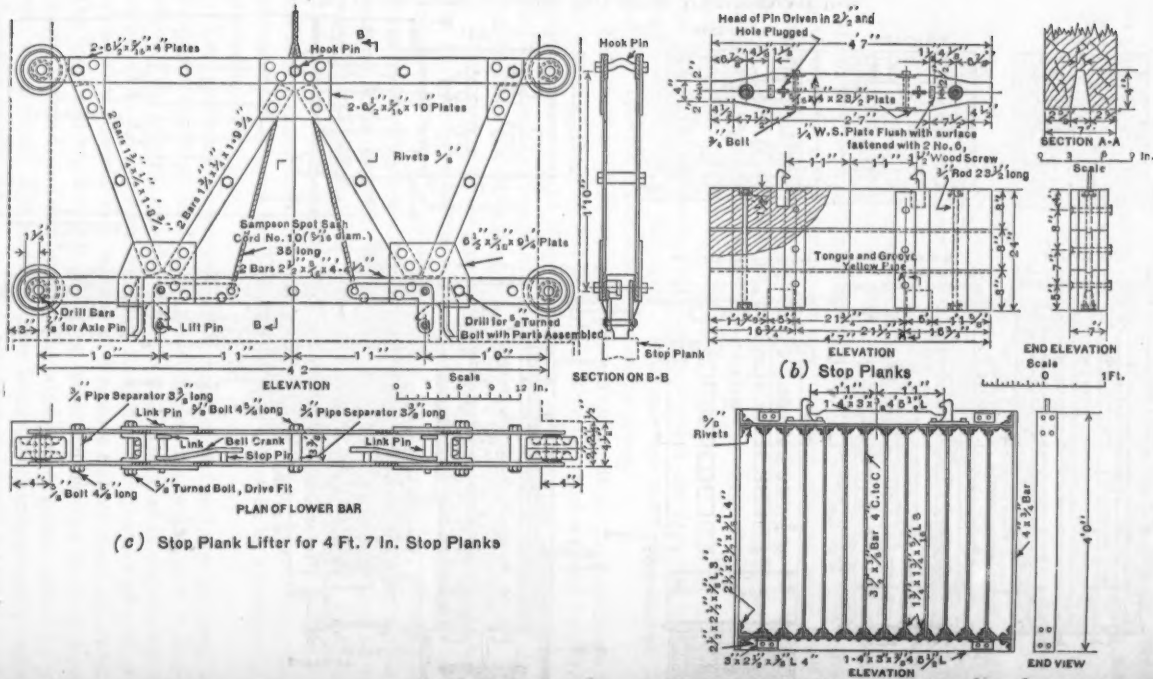

FIG. 20.—SECTIONS OF COARSE BAR SCREEN, STOP-PLANK, AND STOP-PLANK LIFTER FOR 4-FOOT 7-INCH STOP PLANKS.

Power Gate-House

At the west end of the spillway is a power gate-house, Fig. 21. This was built so that, should the city, in the future, wish to develop power at certain times of the year, that is, with reservoir full and water overflowing the spillway, or when water is drawn from storage, say, from the upper 10 or 15-ft. depth in the reservoir, it could do so by constructing a hydro-electric power plant below the dam, on the west side of the river.

Water would be drawn through the gate-chamber into two sets of wells, which, in turn, connect with a conduit, 15 ft. wide and 19 ft. high, passing under the bridge to the down-stream side of the dam. From the end of this conduit a head-race could be constructed along the side hill in the rock to a suitable point where penstocks could be extended to the power house. The head-race has not been built, but the upper end has been excavated for a distance of a few feet from the conduit. This short stretch of excavation has been roofed over with a reinforced concrete slab, above which is a covering of loam and grass sodding. On the up-stream side of the gate-chamber an approach channel has been excavated in the rock, the bottom of the channel being 20 ft. below the crest of the dam.

The two wells are fitted with cast-iron frames for the future installation of sluice-gates and also with cast-iron grooves for screens and stop-planks. The upper end of each well is closed at present with a 12-in. reinforced concrete bulkhead keyed into the side walls of the well.

The power gate-house is slightly smaller in size than, but of the same construction as, the outlet gate-house.

Pavilion

To balance the structure as a whole, a pavilion has been built at the east end of the spillway. It is similar in construction and dimensions to the power gate-house, except that the wall panels between the pilasters have been omitted on all four sides. In the three sides other than that next to the roadway, balustrades, similar to those on the bridge, have been built, both for protection and architectural effect. From the pavilion a commanding view is obtained, especially up stream across the reservoir.

Spillway Channels

With the width of spillway adopted it was necessary to provide spillway channels below the dam on each side of the river. This was accomplished by excavating benches in the rock, the benches being curved in plan and sloping down stream from the toe of the apron until the river bank is reached. These benches vary in width from a minimum of 32 ft. 11 in. to a maximum of 103 ft. 3 in., and in height from a minimum of 5 ft. to a maximum of 20 ft. In excavating these spillway channels a double purpose was served; not only do they provide for guiding flood-water down to the river, but they also furnished most of the rock for the construction of the dam.

With water passing over the spillway of the dam it flows along these benches and, at the same time, spills over from one bench to the next lower

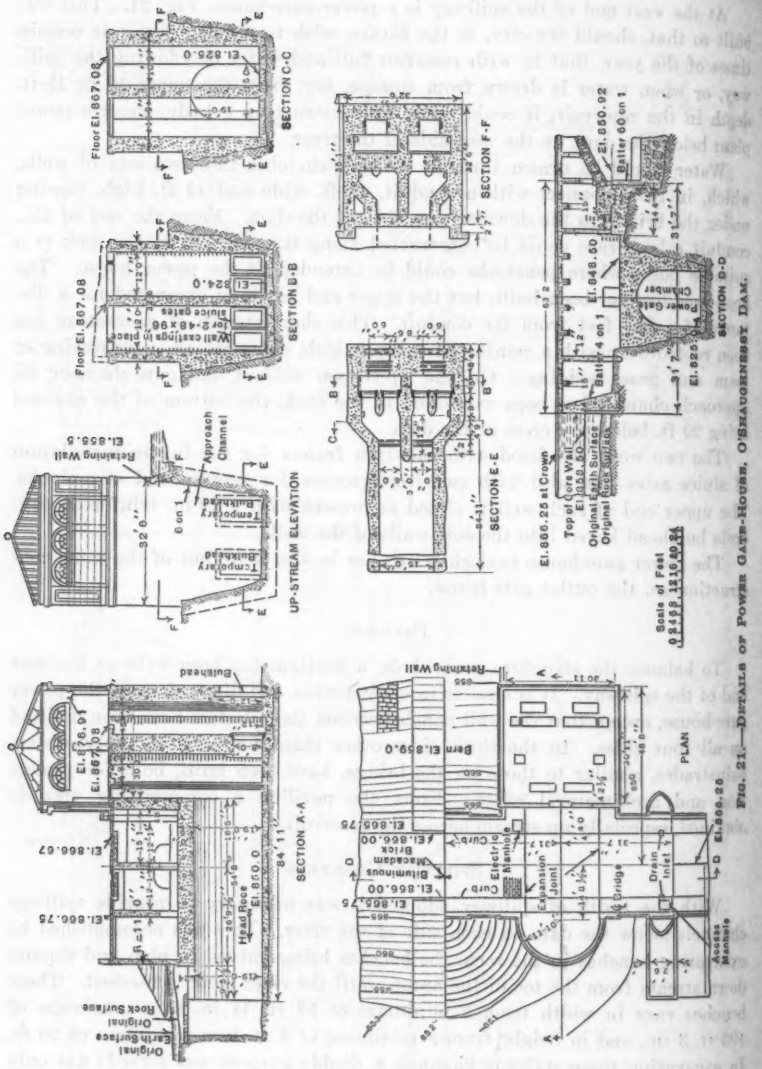

FIG. 21.—DETAILS OF POWER GATE-HOUSE, O'SHAUGHNESSY DAM.

one until it reaches the river. The greater the flow the longer the travel of the water longitudinally of the benches. With any considerable volume of flow over the dam, it is a beautiful sight to see the water rush along the spillway benches and tumble over from one bench to the next. To the present (October, 1927), the greatest depth of water passing over the spillway, on March 21, 1927, was 4.2 ft., corresponding to a flow of about 21 500 cu. ft. per sec. A view of the east spillway channels is given in Fig. 17.

Construction Plant and Methods

For various reasons a more or less elaborate construction plant was required for building the dam. For example, excavated rock, when suitable, and as needed, was to be used as construction material; equipment, other than spouting, had to be installed for transporting concrete as the specifications provided that "spouting of concrete as a means of transportation will not be permitted"; and, again, as the work was located about three miles from the nearest railroad siding, construction equipment, supplies, and building materials, other than those obtained at the site, had to be transported over this distance. It was these, together with other, factors, which determined the type of equipment as well as the methods used in constructing the dam.

Electrically Operated Equipment.—With the exception of the steam shovel, cableway engines, channeling machine, gasoline locomotives, and motor trucks, practically all the plant and equipment used was electrically operated. As there were no power lines in the vicinity of the dam, a 7-mile high-tension line, carrying a 15 000-volt, 3-phase, 60-cycle current, was constructed, connecting the lines of the Columbus, Delaware, and Marion Electric Railroad with a transformer station located near the dam. From the transformer station a 440-volt circuit was run to the crushing plant and 220-volt circuits to the concrete mixing plant, to the air compressor plant, to pumps, and to small motors and lights located here and there on the work and in the camp. About twenty-five motors, of 5 h.p., or more, were in service at various points.

Cableways.—For constructing the masonry part of the dam two traveling cableways were installed, both traveling on the same tracks and having similar equipment and dimensions. The cableways had a span of a little more than 1100 ft. and the towers a height of 103 ft. 6 in. They were equipped with Lidgerwood steam cableway engines and marine type boilers and with electric motor drive for lateral travel of the tail towers. The main cables, lock strand, were 2¼ in. in diameter. On both banks of the river the railroad trackage under the cableways was long enough to permit the up-stream cableway to reach a few feet up stream from the up-stream face of the outlet gate-house and the down-stream cableway to reach the down-stream face of the longest apron. Each cableway had a nominal capacity of 10 tons, but this load was frequently exceeded, as the weight of a full bucket of concrete was something in excess of 12 tons.

Transportation Equipment.—At the beginning of the work the transportation equipment consisted of a fleet of fifteen old White dump trucks; these were supplemented later by a varying number of locally owned trucks, two gasoline locomotives, trackage, and cars. On the east bank, immediately in

FIG. 21.—DETAILS OF POWER GATE-HOUSE, O'SHAUGHNESSY DAM.

front of the cableway towers, two tracks were laid, one running to the concrete plant and the other to the crushing plant. Concrete was hauled, on one track, to either of the cableways in 5-yd. bottom dump buckets set on flat cars; the other track was used for carrying rock from the cableways to the crushing plant.

Trucks were used for hauling all the construction plant and equipment, as well as supplies and building materials, from the railroad spur at Powell, Ohio, 3 miles away; for hauling all the earth excavated, except a small portion used in the coffer-dams; and for hauling a considerable portion of the excavated rock either to the crushing plant direct, or to a point where it could be handled by the cableways. For the maintenance of the trucks a fairly well equipped garage, with repair parts and a few small machine tools, was built.

Excavation.—The bulk of the excavation, both earth and rock, was made with a 1-yd. Osgood steam shovel, mounted on a caterpillar truck. The excavated material was loaded into trucks or into skips handled by the cableways. The cableways delivered the excavated rock to cars on one of the tracks running in front of the cableway towers, or to a grizzly on the east bank where the larger stones were separated, for use in cyclopean masonry, before being discharged into the cars. The cars, in turn, delivered the excavated rock to the crushing plant.

The limestone encountered required extensive drilling and blasting. The drilling of holes for blasting and the drilling of all grout holes less than 15 ft. in depth, was done with jack-hammers driven by compressed air supplied from a stationary motor-driven compressor plant in which were located two, belt-driven, horizontal, Ingersoll-Rand air compressors. A Sullivan channeling machine was used to cut the sides of the cut-off trench, as required by the specifications. The mucking of the rock in the cut-off trench, and in the core-wall trenches, and the trimming of the excavation was all done by hand supplemented by light blasting. Excavation, both hand and machine, was carried on day and night during a large part of the 1922 and 1923 working seasons and during a part of the 1924 season.

Crushing and Screening Plant.—The principal items of equipment installed in the crushing and screening plant were a No. 6 and a No. 25 Kennedy gearless crusher, driven by 40 and 25-h.p. motors, respectively; a rotating cylindrical stone screen, 5 ft. in diameter and 19 ft. long, with a dust screen of larger diameter, 6 ft. long at the inlet end; a four-compartment storage bin and skip hoist; and bucket elevators and belt conveyors, all the equipment being motor-driven. Later, a small rotary screen was installed and used for washing the smallest size stone. The crushers were set at ground level and the screen at the top of the storage bin. The rock to be crushed was dumped into a receiving pit at ground level, hoisted by the skip to the charging platform over the crushers, put through the larger crusher, hoisted to the screen, and screened. The screened material went into bins from which it could be loaded into trucks or to storage piles, the smallest size aggregate being passed through the washer if desired. Tailings from the screen were conveyed to the smaller crusher, were recrushed, and passed through the screen again, together with the crushed stone from the larger crusher. The crushed stone

was screened into four sizes, screenings and dust less than ¼ in., ¼ to ¾ in., ¾ in. to 1½ in., and 1½ to 3 in.

Storage of Crushed Stone and Sand.—The storage plant for crushed stone and sand consisted of a timbered tunnel, with the top at ground level, extending from the concrete mixing plant to a point about 400 ft. distant from the plant; a timber trestle built directly over the first 225 ft. of the tunnel; a Green, portable, belt-conveyor unit; belt conveyors on the trestle and in the tunnel; and a bucket elevator extending from the end of the tunnel conveyor to the top of the concrete mixing plant. Crushed stone, more than ¼ in. in diameter, was delivered from the storage bins under the screen, by conveyor and elevator, to the trestle conveyor which, in turn, discharged it into storage piles over the tunnel and on the ground on each side of the tunnel, the three sizes of stone being kept separate. Sand, hauled by truck from the railroad, was dumped into the receiving end of the portable conveyor unit and was discharged into a storage pile at the outer end of the tunnel beyond the trestle. The sand and crushed stone were discharged through gates in the top of the tunnel and delivered to the concrete mixing plant by belt conveyor and elevator.

Unloading Plant at Railroad Siding.—Bulk material and supplies, principally sand and coal, were unloaded at Powell, on the Hocking Valley Railway, by an unloading plant consisting of a track feeder and bucket elevator and elevated bin from which the material was discharged directly into motor trucks. The equipment was driven by a gasoline engine.

Concrete Mixing Plant.—Practically all the concrete was mixed at a central mixing plant located about 100 ft. up stream from the dam and a short distance east of the east abutment section. The upper part of the plant consisted of four compartments or storage bins for sand and the three sizes of crushed stone; below were two floors above the ground level, the proportioning devices being located on the upper floor and the concrete mixers on the lower. The ground story was a pipe shop.

The equipment consisted principally of two, 1-cu. yd., Smith, tilting mixers; a measuring hopper over each mixer; a collecting hopper, with 2-way chute for feeding the measuring hoppers; four sets of weighing devices, one each for the three sizes of stone and one for the sand; a calibrated water barrel, with 2-way discharge piping; an 8-cu. yd., bottom dump, concrete hopper, in front of the mixers and over the railroad track running under the cableways; and an electrically operated platform elevator for hoisting cement.

Each weighing device consisted of a weigh hopper with a lever-dumping, self-closing bottom; a motor-driven, steel apron conveyor for feeding material to the weigh hopper from an opening in the floor of the storage bin above, from which both were suspended; and a scale-beam with an electric contact so arranged that the lifting of the beam would shut off the conveyor motor. One switch started any one or all of the conveyor motors. The bottoms of the weigh hoppers were connected to a single dumping lever so that, with the scale-beams set, the pressing of the starting switch-button was all that was necessary to weigh out any desired quantities of the three sizes of stone and

of the sand. The pulling of the dumping lever discharged these materials into either of the two measuring hoppers, where the cement was added. The cement was handled direct from and measured as in the original packages.

The specifications required the proportioning of the aggregates by volume. As might be expected, the weight of material for a given volume was not always constant, hence, in order that the materials might be proportioned by weight, which the contractor requested, weighed quantities of the several classes of material were checked, several times during a run, against the volume as measured in the charging hoppers, which were calibrated. If any departure from the required volume was found the scales were correspondingly corrected.

Pumps.—The dewatering of the coffer-dams was done by two, 4-in. discharge, motor-driven, low-lift pumps. When once pumped down the intermittent operation of one pump would handle the leakage. Water for concrete, stone washing, and other plant uses was furnished by two, 3-in. discharge, motor-driven, two-stage centrifugal pumps, one pump being able to supply the demand when the washer was not being operated.

Camp.—The camp included the engineers' office, built for the city by the contractor, as required by the specifications; two contractors' offices; two store buildings; a well equipped emergency hospital, with a graduate nurse in charge day and night; a blacksmith shop; the garage already mentioned; an air-compressor building; a small saw-mill and tool storage building; a large mess hall and commissary; a bath house and latrine; and a varying number of one-story bunk houses, nine being constructed early in the work. All but four of the bunk houses were either torn down or used for other purposes, there being only a few occasions when four were filled. Most of the labor, both skilled and unskilled, drove to the work from the surrounding countryside, or from the dozen or more villages and small towns within a radius of 15 miles, a few making the trip daily from Delaware and Columbus, which were somewhat farther distant.

All the buildings, except the garage and air-compressor buildings, were of frame construction; these two had corrugated iron siding and roofs. The buildings were lighted by electricity and the bunk houses, latrine, and commissary were well ventilated and screened. The engineers' office was the best equipped building in the camp, being provided with a furnace, Delco lighting and power outfit, a drilled well, motor-driven pump, a well-equipped wash and toilet room, and a fireproof concrete vault for the storage of note-books, drawings, and other records.

Sanitation.—Camp wastes were handled in several ways. The large latrine, and smaller ones at several points on the work, and the mess-hall kitchen were provided with galvanized cans which could be removed, through doors at ground level, from the outside of the buildings. All wastes so collected were buried in pits dug at some distance from the work, the contents of the pits being covered with quicklime and a few inches of earth as soon as deposited. Wash water from the latrine and kitchen was run into farm tile drains from which it was allowed, by sub-surface irrigation, to leach into the ground. The engineers' office was provided with a small sewage treatment plant, consisting

of a covered circular concrete septic tank and a sub-surface irrigation system of two units. Each unit had a length of 232 ft. of vitrified pipe, wrapped with burlap at the joints and surrounded with crushed stone. Drinking water, other than that for the engineers' office, was obtained from a 100-ft. well by a motor-driven pump and stored in an elevated wooden tank erected close to the mess hall to which it was piped.

Other Plant and Equipment.—Mention should also be made of the cement storage house, 60 ft. wide by 100 ft. long, with a capacity of about twenty carloads of cement, a stiff-leg derrick with an 80-ft. boom, used for many purposes and at several points on the work, the road-building machinery, a complete compressed-air operated drill dressing outfit, and a grouting machine.

Coffer-Dams

In designing the dam it was obviously desirable to take full advantage of the topography of the top surface of the rock, as developed by the test borings, and to make the spillway as long as possible, in order to keep at a minimum the depth of water which would have to pass over the spillway under the assumed maximum flood flow. It was decided, therefore, to place the ends of the spillway section at about the points where the rock surface reached the elevation of the crest of the spillway. This brought the center of the spillway close to the low-water mark on the east side of the existing river bed, at which point the rock outcropped and there was no question, either from surface indications or borings, as to the suitability of the location for the outlet gate-house.

As previously stated, provision was made in the plans for placing four 48-in. pipes in the up-stream part of the gate-chamber to take care of the ordinary flow of the river during the construction period. The construction of the gate-house was, therefore, the first work started in the river bed. The first coffer-dam, on the east side of the river, was begun in the early part of September, 1922, before the completion of the general construction plant. It was planned by the contractor to extend the coffer-dam to Elevation 790, 9 ft. above low water, but on the recommendation of the engineers the height was reduced to about Elevation 784, roughly 3 ft. above low water in the river, as coffer-dams of this height had proved satisfactory in the construction of the Julian Griggs Dam.

In addition to that section of the dam in which the outlet gate-house is located, sufficient area for the next section to the west, out into the river, was included in the east coffer-dam, the inside dimensions of which were 175 ft., measured at right angles, and between 145 and 150 ft., measured in a direction parallel to the dam. For the construction of the up-stream, down-stream, and river sides of the coffer-dam, cribs, about 10 ft. in width, made of small logs secured on the site of the work, were built, the framing being done on the bank of the river. When complete, the cribs were skidded into the river, floated into position, fastened together, and sunk by filling them with rock. Earth was then deposited around the outside of the coffer-dam, after which 2-in. sheeting was placed, edge to edge, all the way around the outside face of the cribbing. The sheeting was held at the top by waling timbers

attached to the cribbing, and was driven into the earth by mauls. As the water was pumped down in the coffer-dam additional earth and small rock was placed in the cribbing and outside the sheeting, the outside fill being compacted by men, with rubber boots, until the top of the fill was above the water level in the river. In spite of the fact that the entire easterly part of the river bed was bare ledge rock, a remarkably tight contact was secured.

The first construction work in the coffer-dam was started on October 6, 1922, and continued until December 28, when the coffer-dam was topped by a moderate flood. An attempt was made to pump the water out during the early part of the following month, but this had to be abandoned on account of another rise in the river. It was then decided to cease operations until spring. Some trouble was encountered during the winter with ice jams, but only one actually did any damage. A view of the east coffer-dam, taken on November 29, 1922, is given in Fig. 22.

On May 1, 1923, the coffer-dam was pumped out and construction work resumed, but it was again flooded on May 12, and it was not until May 24 that construction work was again started. On June 6, and again on June 15, heavy local showers raised the river sufficiently to flood the work for an hour or two, but operations were resumed the following day.

On June 25, 1923, an opening was made in the east coffer-dam and the river turned into the 48-in. pipes passing through the gate-chamber. On the same day the up-stream portion of the coffer-dam on the west side of the river, previously constructed, was closed. The up-stream side of the west coffer-dam was similar in construction to the east coffer-dam, but was tied into a concrete wall, constructed for the purpose, which projected up stream several feet from the westerly end of the concrete already in place in the east coffer-dam. The river side of the west coffer-dam was also of the same construction, the cribbing resting on the concrete apron, which was already in place, and tying into the westerly end of the down-stream cribbing of the east coffer-dam. The down-stream side of the west coffer-dam consisted of an fill made of earth excavated from the west bank of the river. The up-stream side of the west coffer-dam was also reinforced by dumping excavated material on the river side of the cribbing, raising the effective height to about Elevation 788. The area enclosed by the west coffer-dam was, roughly, 130 ft., up and down stream, by 180 ft. parallel to the axis of the dam.

The west coffer-dam was flooded twice, first, on July 8, 1923, when a part of the down-stream fill was washed out and had to be replaced, and the second time on September 22, when little damage was done. On November 2, 1923, the concrete work was completed to a point above low-water level. From this time, however, the up-stream side of the coffer-dam continued to be used as a means of crossing the river, a timber foot-bridge spanning the opening through which the river flow passed to the 48-in. pipes in the gate-chamber.

Excavation

Before describing the methods used in making the excavation for the dam and appurtenant work it will be well to state briefly the condition existing at the site. The Columbus or Delaware limestone was encountered at all

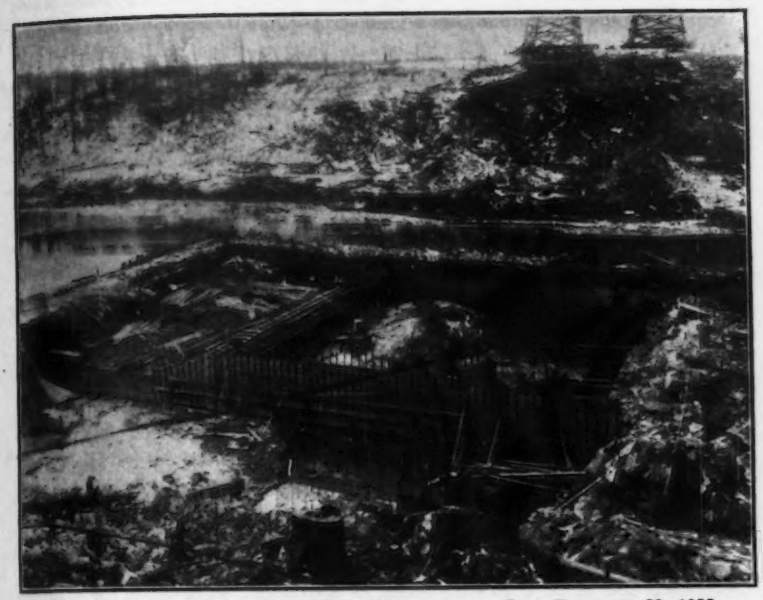

FIG. 22.—VIEW OF EAST COFFER-DAM, O'SHAUGHNESSY DAM, NOVEMBER 29, 1922.

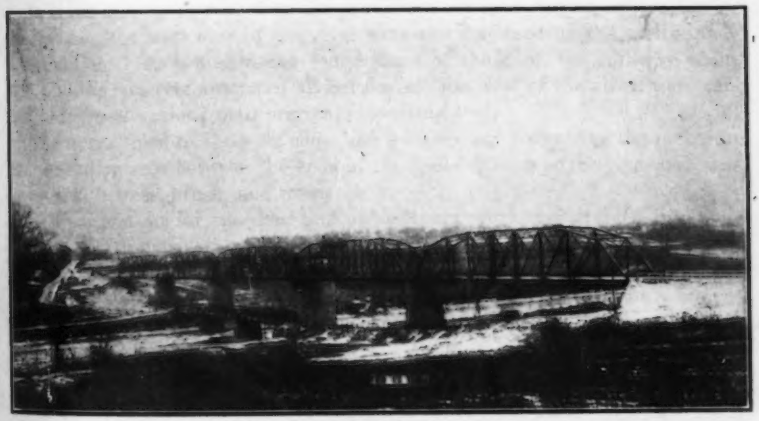

FIG. 23.—RATHBONE BRIDGE—THE OLD AND THE NEW.

points under the dam and core walls, the cover varying, however, both in character and thickness, the latter reaching a maximum of about 15 ft.

The higher lying banks outside of and at each end of the concrete part of the dam were cultivated areas with 2 to 12 ft. of cover consisting of clay loam, yellow boulder clay, and a varying thickness of clay mixed with rock fragments immediately above the ledge rock. The first sharp rise from the river bed on the east bank had only such cover as was held in place by the roots of trees, brush, and weeds. East of this rise was a moderately sloping terrace, the rock being covered with from 2 to 5 ft. of leaf mould, loam, and clay and rock fragments mixed. In this area the top of the rock had been affected by weathering to a greater extent than at any other part of the site except for a few small areas on the west bank. The second sharp rise east of the river was covered with material similar to that overlying the terrace, but of less depth.

The easterly half of the river channel was bare ledge rock, whereas the westerly half was covered to a depth of from 1 to 4 ft. with a bar of dirty sand, gravel, and small fragments of rock. Immediately west of the river was an alluvial terrace from 30 to 50 ft. wide, the cover over the rock running from 4 to 9 ft. in thickness and of material similar to that found in the river, except with a considerable admixture of leaf mould and loam. The lower two-thirds of the steep slope west of the river consisted of rock steps filled, to an evenly sloping top surface, with loam, leaf mould, and clay and rock fragments of all sizes, some measuring as much as 5 or 6 ft. in one dimension. Some of the vertical faces of the steps were 15 ft. high and both the vertical and horizontal faces were more or less weathered, in a few places, to a considerable depth.

It is interesting to note that the conditions revealed during excavation in this area on the west side of the river were not disclosed by the preliminary investigations. As a result, the foundations of the dam, the spillway channels, and the concrete aprons of all but one section west of the river, were carried to lower elevations than originally contemplated.

The upper third of the west slope had scarcely any cover over the rock but, like the entire area between the ends of the spillway, except the river bed, was covered with trees, brush, and weeds.

The excavation for the dam and appurtenant work was divided into four classes: Earth excavation above Elevation 782; rock excavation above Elevation 782; excavation below Elevation 782; and rock excavation in the cut-off trench. The contract prices for the first three classes of excavation were the same, $1.90 per cu. yd., and, for rock excavation in the cut-off trench, $12.00 per cu. yd.

Except for some hand work for the engineers' office and certain small culverts, actual excavation was started on July 22, 1922, on that part of the relocated highway east of the dam where cuts were necessary. This was followed by the stripping of the cover and loose top rock on the terrace east of the river, all the excavated material from these two locations being placed in embankments made necessary by the relocated highway. Excavation for the

east section of the spillway and rough excavation for the east spillway channel came next, followed, successively, by excavation for the sections of dam and spillway channels west of the easterly section, as far as the top of the first steep slope east of the river. Some of the excavated material was used to complete the road embankments, the remainder being placed in storage piles adjacent to the stone crushers up to the time the crushers went into operation. Prior to the completion of the first cableway (there were two), all the hauling of excavated material was done by trucks, and thereafter by both cableway and trucks, the trucks hauling part of the excavated rock direct to the crushing plant and a large part of the remainder to large steel skips which, in turn, were handled by the cableways.

On October 10, 1922, excavation by steam shovel was started in the first coffer-dam, and completed on November 15. On this latter date rough excavation for the remaining sections of the dam and spillway channels east of the outlet gate-house was started. This work was carried on, as weather permitted, during the winter, all the rough excavation for the remaining sections of the dam, east of the river, being completed, as well as a part of the spillway channel excavation, by March 27, 1923, when the shovel was moved to the west side of the river and excavation started for the westerly spillway section.

Excavation on the west bank was carried on, successively, from rock step to rock step, working toward the river, part of the excavated rock being taken to the crushing plant, by cableway and railroad, the remainder being cast over on the next lower section and again rehandled. It was during this period that a second steam shovel was put in operation, on the terrace just west of the river.

On July 14, 1923, steam-shovel excavation was begun in the west coffer-dam, following which came the completion of the excavation on the terrace immediately west of the river. Part of the rock from the coffer-dam was transported to the crushing plant, but the bulk of the mixed earth and rock from the west bank was placed in spoil banks below Elevation 792 on the upstream side of the dam. By the early part of September the steam shovel work for the dam proper was complete, and work was again taken up on the spillway channels. It was necessary to go over the spillway channels at least once and, in some instances, several times to put them in final acceptable condition, this work being partly done during the remainder of 1923 and completed in 1924. The excavation necessary for the new road connecting the west end of the dam with the existing county highways, as well as the excavation of most of the earth used in building the embankment sections of the dam, was made during the latter part of 1924.

A considerable amount of hand excavation was necessary in trimming the area under the dam and apron and the spillway channels. The bulk of the core wall and cut-off trench excavation was hand work supplemented by light blasting of the rock, the sides of the cut-off trench first having been cut with a channeling machine to the full depth. The excavation for the pavilion walls and for the approach channel to and for the head-race below the power gate-house was also done by hand mucking after blasting.

Throughout the work it was necessary to drill and blast the rock before it could be handled by steam shovel, and wherever the excavation had to be brought to neat lines it was necessary to line drill the rock before blasting to prevent overbreaks and to hand trim the faces of the excavation, this being especially the case along the edges of the spillway channels where, even with the precautions noted, some overbreaks occurred. Practically all the rock excavated by hand was carried by the cableways where the work was within the area served by them, and by truck from all points outside this area.

Grouting the Foundation

The information obtained from the core borings made on the site of the dam and from the hydraulic tests made on the cored holes resulted in the decision to grout the foundation in order to close up any open bedding planes, solution channels, cleavage cracks, or fissures that had been located by the borings or tests, or that might exist at points not explored by the borings. The hydraulic tests had further developed the fact that in general the rock was fairly water-tight below a depth of approximately 10 ft. or less below the top of ledge rock, except at a few points which could be given special attention. It was decided, therefore, that in general the grout holes should be drilled to an approximate depth of from 8 to 10 ft. below the top of the rock, after the excavation was completed. The depth, however, was varied by drilling deeper along the edges of the steps in the foundation adjacent to the lower sections of the dam and to a less depth on the other side of the steps, the holes along the side adjacent to a lower section reaching to from 3 to 6 ft. below the top of the rock in that section after the excavation in this section had been completed. It was also decided that there should be a row of holes up stream from the cut-off trench, unless the rock at this location showed no seams or cracks, and a row in the cut-off trench, the holes in these rows to be spaced not more than 12 ft. apart, and also holes on 10 to 12-ft. centers, both ways, down stream from the cut-off trench and under the dam, but not under the apron, and additional holes into every crack and crevice found in the excavated area.

A number of crevices were opened by the excavation, a few at the easterly end of the spillway section and just west of the river bed being filled with earth. Each crevice was cleaned out with water under pressure and grout pipes were placed in the crevice, or holes drilled, so as to intercept the crevice, or both, with the idea of making it possible for the grout to reach every channel through which water might seep under the dam. In several of the spillway sections no attempt was made to follow any regular spacing of holes, the location of the cracks and seams in the rock determining the location and spacing of the holes drilled.

A total of 553 grout holes were drilled under the dam, abutments, and core walls, 526 of this number being drilled with jack-hammers, using a 2-in. bit, the over-run in width at the top of the hole being enough to admit the 2-in. standard weight, wrought-steel pipe used for grouting. These holes ranged from a minimum of 4.1 ft. to a maximum of 11.1 ft. in depth, the average being 9.6 ft. For the holes under the large concrete abutment blocks at either

end of the spillway and under the two westerly sections of the spillway, it was necessary to use a well-drilling machine with a 4-in. spud-bit to reach to a depth lower than the bottom of the adjacent section of the dam. Twenty-seven machine-drilled holes were put down, ranging from 18.0 to 30.5 ft. in depth, the average depth being 26.4 ft.

As soon as the drilling was completed, grout pipes of sufficient length to reach above the first lift of concrete were put in place. In the holes drilled with jack-hammers, 2-in. pipe was used; in the machine-drilled holes, 4-in. pipe was required, the pipe being cemented in, where necessary, to exclude the concrete. Where the elevations of the foundation of adjacent sections of the spillway differed by more than about 8 ft., it was necessary to hold back the grouting of the higher section until part of the concrete had been placed in the lower section, pipe being extended up through any additional lifts of concrete placed in the higher section. In some of the sections at the westerly end of the spillway the pipes were carried through the up-stream face of the dam. In the sections of the spillway in the river bed, it became necessary to limit the length of pipe for which payment would be allowed. The extension of the pipe was permitted, however, in order to expedite the placing of concrete; but the length for which payment was allowed was limited to 4 to 5 ft. under the dam and 11 to 14 ft. in the cut-off trench. For each fitting that was necessary, an allowance of 1 lin. ft. of pipe was made and the figures given subsequently include this allowance.

Excluding the grout-pipe connections to the 48-in. outlet pipes in the gate-chamber, and pipe used for other purposes, a total of 4 363 lin. ft. of pipe was paid for, the minimum length for any one hole being 2 ft. and the maximum, 43 ft. The total length included eight grout-pipe connections to sink holes, springs, and mud seams where no drilled holes were required.

The grouting machine was made on the job from a stock cylindrical steel air reservoir, 28 in. in diameter by 44 in. high, having pressed oval heads, the upper head containing an oval cast-iron clean-out manhole held in place by two pressed steel yokes and bolts. Angle-iron legs were welded to the tank, and a 3-in. outlet pipe, with a 3-in. brass cock and reducer for connecting to 2-in. pressure hose, was screwed into the bottom head. For mixing the grout four ½-in. pipes were screwed through the lower head about 4 in. from the outside shell and on the quarter-points, each pipe having an elbow and short nipple on the inside end and all four pipes being connected to a 1-in. air line running up the outside of the tank. This air line had a 3-way cock, to the outstanding opening of which the air-supply hose was connected, and two tees, one for the pressure-gauge connection, and the other a blow-off, with a valve, which extended through the top head of the tank.

The operation of mixing the grout consisted of filling the tank to the proper point with water, through the manhole, turning in a small quantity of air through the mixing pipes, placing 2 sacks of cement and 2 sacks of sand in the tank, screwing the manhole casting in place, the blow-off being open, and turning on of full air supply to the mixing pipes for at least 1 min.

Connection between the tank and the grout pipes was made with 2-in., rubber-lined linen hose. The grout was applied by closing the blow-off valve

and opening the outlet cock, the air supply being turned in through the top connection if it failed to move the grout when being supplied through the mixing pipes.

Payment for the grout placed was made by the cubic yard which by definition consisted of 8 cu. ft. of cement, 8 cu. ft. of sand, and the necessary water, the actual volume of this quantity of wet grout being somewhat less than 1 cu. yd. A fine brick sand, 45% or more of which would pass a 40-mesh screen, was used. A total of 229 cu. yd. of grout was placed, the quantity in the several sections of the spillway varying widely. The minimum quantity placed in any one section, 71 ft. 6 in. in length, measured along the axis of the dam, was 0.75 cu. yd. The maximum quantity in any one section was 38.63 cu. yd.; and in one hole in this section, 18 cu. yd. were placed, this quantity being the maximum for any one hole. The average quantity per hole, including the sink holes, springs, and mud seams, was 0.41 cu. yd.

The full air pressure carried in the supply lines, of 80 to 100 lb. per sq. in., was applied to the holes, only 50 lb., however, being required by the specifications. If grout rose in any of the holes surrounding the one being grouted, the pipes were capped and the grouting was continued. In one case grout was raised in fifteen holes from one connection. Later, the caps on the pipes were removed; if they were not filled, grout was then pumped into them, the same treatment being applied to all pipes that had taken grout upon the first connection.

The number and length of drilled holes, the amount of pipe and fittings used, and the grout placed considerably exceeded the estimated quantities, but the results obtained have been considered as amply warranting the added cost. A thorough examination of the area down stream from the dam has failed to disclose any leakage other than a slight seepage around the three westerly sections. This leakage is so small as to cause no uneasiness from the possible formation of solution channels through the rock.

Concrete

In the construction of the dam, five classes of concrete were used. For convenience of reference Table 1 gives the mixture used in, the contract price per cubic yard of, the volume placed under, and the location in the work of, each class of concrete. The total volume of concrete placed was 79 565 cu. yd., at an average price of $9.29 per cu. yd.

The specifications permitted the engineer to change the relative volumes of the fine and coarse aggregates in each mixture, but at the same time keeping the total volume of the two classes of aggregate the same. The coarse aggregate was screened into three sizes: No. 1, ¼ to ¾ in.; No. 2, ¾ to 1½ in.; and No. 3, 1½ to 3 in. In most of the work a mixture of the three sizes was used, but in much of the reinforced work the No. 3 coarse aggregate was omitted, and, at times, only the No. 1 coarse aggregate was used, especially in thin reinforced work, in order to ensure satisfactory bedding of the steel reinforcement bars.

The specifications for the fine aggregate provided that the material should contain not less than 80% of silica, graded from fine to coarse, should be

thoroughly screened to reject all particles greater than ¼ in. in diameter, and should not contain more than 3% of material passing a 100-mesh standard sieve; also, that the material should not contain more than 3% of loam, clay, or dirt. The fine aggregate used was a washed bank sand shipped from Dresden, Ohio, distant about 55 miles east of Columbus.

TABLE 1.—CONCRETE USED IN THE O'SHAUGHNESSY DAM.

Class.	Mixture.	Contract price per cubic yard.	Volume placed, in cubic yards.	Location.
A.....	1 : 3 : 6*	$ 6.85	56 189	Cyclopean masonry forming the body of the dam and bridge piers.
B.....	1 : 2½ : 5	13.00	2 032	Core walls of the earth dikes; retaining walls and culverts.
C.....	1 : 2 : 4	14.00	17 362	Exposed faces of dam and bridge piers; cut-off trench; substructures, not including floors of gate-houses and of pavilion; paving and retaining walls in regulating works below dam.
D.....	1 : 2 : 4	21.00	4 002	Bridge arches, spandrel and curtain-walls; floor of bridge, including sidewalks; floor of gate-chambers, pavilion, and balconies; and bulkheads in power tubes.
E.....	1 : 2 : 4	40.00	84	Roof decks for superstructures of gate-houses and pavilion.
Average.....	.....	$ 9.29	.....	....
Total.....	.....	.....	79 569	....

* Not including the embedded stones.

The coarse aggregate was crushed limestone obtained mainly from the rock excavation for the dam and spillway channels and was screened on the job into the three sizes mentioned.

In the 1 : 3 : 6 Class A concrete, designated as cyclopean masonry and forming the main body of the dam and piers of the bridge over the spillway, large stones, from the rock excavation, were embedded to a certain extent, but the percentage of such stones, taken as a whole, was small.

Alpha Portland cement was used throughout the work, the total amount being 93 500 bbl. With a total volume of 79 569 cu. yd. of concrete placed in the dam, the ratio of cement to concrete was 1.18 bbl. of cement per cubic yard of concrete.

The proportioning of the relative volumes of fine and coarse aggregate and of the three sizes of coarse aggregate was adjusted throughout the course of the work with the object of securing concrete of maximum density.

In all the concrete very little departure, in general, was made from the specified proportions of fine and coarse aggregate. When three sizes of coarse aggregate were used, the proportions were, in general, 1 volume of No. 1, 1 volume of No. 2, and 2 volumes of No. 3 aggregate, and when only two sizes were used the proportions were, generally, 1 volume of No. 1 and 2 volumes of No. 2.

Expansion Joints

In the spillway section of the dam, under each arch of the bridge, an expansion joint was located. At each of these expansion joints, in addition to the so-called bonding grooves, Fig. 14, a copper strip, $\frac{1}{8}$ in. thick and 7 in. wide, was inserted across the down-stream side of the up-stream bonding groove. The copper strip was so placed as to be free to move with the contraction, and subsequent expansion, of the adjacent sections of the dam.

The concrete roadway slab, carrying the brick pavement, and the concrete sidewalks had expansion joints at the third points of each arch span, at the center lines of the walls forming the upper extension of the bridge piers, and at suitably placed locations opposite the gate-houses and pavilion. In the sidewalks and curbs, Carey "Elastite" was used in the expansion joints.

Expansion joints were also provided in the Indiana limestone balustrade on the bridge as close to all sidewalk joints as the design would permit, no mortar being placed between the stones at these points, the upper rail being held by 1-in., brass dowel pins, 6 in. long.

Construction of Earth Approaches

At each end of the spillway section the approaches were built of earth embankments, with a core wall in the center of each. In the construction of the embankments the surface of the ground was first stripped and all perishable matter removed, as well as loose stones and boulders. The embankments were made of yellow boulder clay, obtained from borrow-pits, and were free from perishable matter and from stones of more than 3 in. in their greatest dimension. On the up-stream side of the embankments, selected material was used which would compact and be as nearly water-tight as possible. On the down-stream side the requirements as to the materials were less exacting and in certain parts some small stone was placed near the outer face, but not in pockets, and back from the face to allow a sufficient thickness of earth covering. Frozen materials were not permitted to be used, nor were embankments built in freezing weather.

The materials were spread in approximately horizontal layers not exceeding 4 in. in thickness and were compacted by rolling with power-operated grooved rollers weighing not less than 2 000 lb. per lin. ft. of width and not less than 4 ft. wide. If the materials were not sufficiently moist when spread, they were moistened with water so as to permit of proper compacting.

The core walls in the approaches began at the outside of the bridge abutments and extended away from each bridge abutment to a point 20 ft. beyond the point where Elevation 858.5 met the original surface of the ground. They were carried down 12 in. into solid rock.

The bituminous macadam roadways on the approaches connect with the bridge over the spillway and with the relocated roads built to connect with existing county highways. A typical section through one of the earth approaches is shown in Fig. 18.

The Reservoir

The O'Shaughnessy and Julian Griggs Reservoirs are quite similar in general characteristics, in that both are long and narrow, the main differences being that the O'Shaughnessy Reservoir is deeper, somewhat longer, and impounds about three times as much water.

The total length of the O'Shaughnessy Reservoir is 8.0 miles. At Elevation 845, normal water level, the reservoir has an average width of 855 ft.; the maximum width is about 1 900 ft., about 0.4 mile above the dam; the minimum width is about 200 ft., at Bellepoint, where the river widens and forms the upper end of the reservoir.

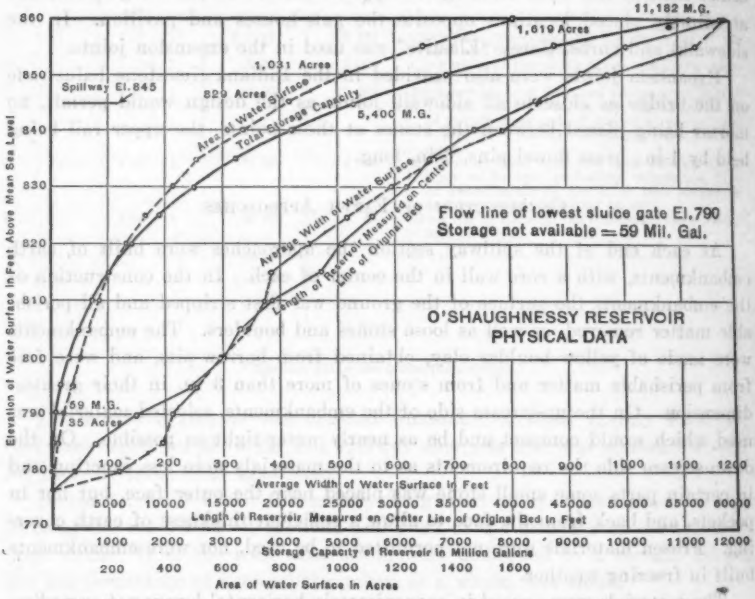

FIG. 24.—O'SHAUGHNESSY RESERVOIR, CAPACITY, AREA FLOODED, LENGTH, AND AVERAGE WIDTH AT VARIOUS ELEVATIONS.

The area of the water surface at Elevation 845 is 829 acres, or 1.57 sq. miles. For sanitary, as well as for general control, a wide marginal strip was acquired by the City on each side of the reservoir. On the west side, this strip has an average width of 227 ft.; on the east side, the average width is 278 ft.; the greater width on the east side being due to the fact that the City acquired all the land between the reservoir and a new highway built to replace one which is now submerged. On the west side the marginal strip has an area of 204 acres, and on the east side, 241 acres, the total area of the marginal strip on both sides of the reservoir being 445 acres, or equal to about 54% of the area of the water surface of the reservoir. The total area of the reservoir and the marginal strip combined, is 1 274 acres, or about 1.99 sq. miles.

On the west side of the reservoir, about 2.5 miles above the dam, at the Girls Industrial School, a correctional institution of the State of Ohio, and for a distance of about 3 200 ft., no marginal strip was acquired, as the land is owned by the State and the City lacked the power to acquire the property except by purchase, and the State did not care to sell. The City, however, did secure the right to flood the State-owned land, along the river, so as to be able to construct the reservoir.

The reservoir has an average depth of 19.8 ft., the maximum being 69 ft., just above the dam. The total volume impounded at Elevation 845 is 5 400 000 000 gal.; of this, 5 341 000 000 gal. are available, taking low water at Elevation 790, or 55 ft. below the crest of the spillway.

The capacity of the reservoir and the area flooded, as well as the length and average width at various elevations, is given on Fig. 24. Table 2 gives comparative data of the O'Shaughnessy and Julian Griggs Reservoirs.

TABLE 2.—COMPARATIVE DATA OF THE O'SHAUGHNESSY AND JULIAN GRIGGS RESERVOIRS.

	O'Shaughnessy Reservoir.	Julian Griggs Reservoir.
Length, in miles	8.0	5.8
Width, in feet:		
Average	855	516
Maximum	1 900	830
Minimum	200	200
Area of water surface, in acres	829	363
Depth below crest, in feet:		
Average	19.8	14.5
Maximum	69	35
Storage, in gallons*:		
Total	5 400 000 000	1 720 000 000
Available	5 341 000 000	1 487 000 000

* By the addition of flash-boards to the Julian Griggs Dam, an additional storage of 500 000 000 gal. can be secured in the Julian Griggs Reservoir.

CLEARING AND GRUBBING

Notwithstanding the fact that the water is purified by filtration, it was considered advisable to grub as well as to clear the reservoir site, as was done in the case of the Julian Griggs Reservoir. Clearing and grubbing, therefore, was carried out over the whole of the area flooded and up to an elevation 2 ft. above normal water surface, the total area so cleared and grubbed being 741 acres.

All trees, saplings, and brush were cut down flush with the surface of the ground, and all stumps and roots larger than 2 in. in diameter were removed to a depth of at least 6 in. below the surface of the ground. In removing stumps both dynamite and stump-pullers were used. Prior to clearing and grubbing, property owners from whom land was acquired, were permitted to cut down and remove such of the timber as they desired.

On the west side of the reservoir the property acquired by the City was not bounded by a highway, and line fences and stone walls were left in place, the City permitting abutting owners, for the present, to water stock at the

reservoir edge. On the east side of the reservoir, where the land acquired is bounded by a new highway, all fences were removed and stone walls leveled. All material removed in the clearing and grubbing operations became the property of the contractor and was either burned or removed to points outside the limits of the property owned by the City. Material was not disposed of by throwing it into the river and allowing it to float down stream. After the clearing and grubbing had been completed, the entire ground surface was burned over, and, prior to filling the reservoir, where grass, weeds, and vegetation had grown up subsequent to the first burning over, the ground was burned over a second time.

The removal of buildings was not included as part of the work of clearing and grubbing, arrangements having been made with some property owners for the removal of their buildings, whereas the other buildings were sold, the prices paid by the purchasers including their removal.

Relocation of Roads

From the dam to the upper end of the reservoir, on the east side, there were county roads running, in general, parallel to the river. For much of the distance these roads were at such an elevation that they would be flooded when the reservoir was filled, and it was incumbent on the City, therefore, to build a new road to replace those flooded. On the west side of the reservoir it was necessary to build a short stretch of new road where an existing road was flooded, above and below Eversole Run, one of the tributaries of the river, and also to build a second short stretch of road from the west end of the dam to an existing north and south highway. At the east end of the dam, the earth section started, practically, at the new road. Where new roads were built arrangements were made with the County Commissioners of Delaware County whereby the City was to acquire the land for and build the new roads and then deed them to the County, the County, in turn, to deed to the City the old roads which would be flooded.

The old roads were mainly gravel roads, in some places nearly impassable in winter. In replacing these roads the City built water-bound macadam highways conforming, in general, to the standards of the Ohio State Highway Department for similar conditions. The macadam surface was 16 ft. in width and had a finished thickness, after rolling, of 6½ in. Crushed limestone, obtained locally, was used. With the exception of the Rathbone Bridge, mentioned later, and the bridge over the dam, the macadam surfacing was carried continuously over the reinforced concrete bridges spanning the small streams tributary to the reservoir. A total length of 7.5 miles of new road was built by the City. In 1926, the new road on the east side of the reservoir was taken over by the State of Ohio and incorporated as a part of the State Highway System, and it has been improved by widening the surfacing and making it a bituminous-surfaced, water-bound macadam.

Reconstruction of Bridges

Mention has already been made of the replacement of the Powell Bridge by the reinforced concrete arch bridge over the dam. At the Girls Industrial

School, about 3.1 miles above the dam, the river was spanned by a bridge known as the Rathbone Bridge. With the filling of the reservoir this bridge would have been entirely submerged. As the bridge was on a main east and west highway, a crossing of the reservoir at this point could not be abandoned.

The superstructure of the existing bridge consisted of three 100-ft. steel trusses. These trusses were in good condition and were suitable for use in a new bridge, if reinforced. Wishing to make use of these trusses in the new bridge if economically possible, the specifications therefore were drawn so as to permit a bidder to submit a proposal for raising the three trusses a total height of about 35 ft. and of moving them on to new piers, and then lengthening the bridge by the addition of eight new 100-ft. trusses of similar construction, or of building an entirely new bridge.

The low bid received, on the basis of using the three old trusses, was $119 240. The low bid for an entirely new bridge, with 16-ft. roadway, was $115 968, and $120 718 with an 18-ft. roadway. The City accepted the bid for an entirely new bridge, with the 18-ft. roadway.

The superstructure of the new Rathbone Bridge consists of five steel trusses, each 207 ft. 6¾ in. in length. The piers and abutments are of concrete. The bridge has a total over-all length of 1 040 ft., face to face of end abutments, and it is the longest bridge in Delaware County. The lower clearance line for the bottom chord and floor members is 13 ft. 6 in. above normal water level in the reservoir. The entire structure was built in accordance with the requirements of the 1918 Specifications for Highway Structures, Class T-15, of the Ohio State Highway Department. A view of the old bridge and the new is shown in Fig. 23.

On the west side of the reservoir at Eversole Run, previously mentioned, a small reinforced concrete bridge was built in connection with the relocation of a part of one of the county roads, and on the east side of the reservoir along the new road several small culvert bridges were built.

Moving Church and Cemetery

Near the upper end of the reservoir was the Scioto Valley Christian Union Church and the Oller Cemetery. The church and cemetery served a rural population and had been there for years. With the flow line adopted for the reservoir, it was found that a part of the cemetery would be submerged and that the church would either have to be moved or a new church built.

Following a series of conferences with the Church Trustees it was decided to move the church. This was accordingly done, the City acquiring and deeding to the Church Trustees the new piece of property on which the church is relocated. The church was a small brick building, 31 ft. wide by 39 ft. long, with 8-in. walls; it was moved horizontally a distance of 350 ft. and, at the same time, raised a distance of 14.2 ft., vertically. To avoid cracking the walls, while being moved, a heavy steel band was placed around the building.

When it comes to moving a cemetery, however small, the question of sentiment has to be taken into account. A happy solution of the problem, however, was found when the Cemetery Trustees agreed to supervise and do all the

work, the City paying all the bills for labor and material. All told, ninety-nine re-interments were made.

Reconstruction at Girls Industrial School

At the Girls Industrial School, a very considerable amount of reconstruction work was necessary. In fact, barring the construction of the dam proper, it was the one most expensive item involved in building the reservoir. The institution, in addition to housing the necessary State officials and employees, houses between 600 and 700 inmates, and for years had been in its present location on the west bank of the river. It had been expanded from time to time and new utilities and conveniences added, some of which were not far from, and only slightly above, the river. The best site for the dam was down stream from the school, but the adoption of that site involved flooding State property, buildings, and utilities. The City had no power to condemn State property, so it was faced with the situation of either securing permission from the State to flood some of the property owned by the State, and replace the buildings and utilities which would be wholly or partly submerged, or of constructing the dam on a very much less desirable site above the school. The City decided to seek permission from the State to flood State property and bear the necessary cost of replacements. To secure this permission it was necessary to secure an Act of the Legislature. Such an Act was passed, putting the matter, in so far as the State was concerned, in the hands of the Department of Public Welfare and authorizing the Director of Public Welfare to make the necessary arrangements with the City. The replacements made by the City included the building of a new power house, laundry, and sewage disposal works, together with the necessary connections thereto, and, after these replacements were made, they were turned over to and became the property of the State.

Power House.—The power house was a short distance from the river and supplied the institution with light, heat, and water. The building was old and in need of repair, and some of the equipment was only in fair condition. The site had been poorly chosen, as the power house had been flooded in the past with high water in the river. The City built for the institution a new brick power house at a much higher and more convenient location.

Such of the existing equipment as could be moved, was transported to the new building and put in running order. The principal items of equipment moved were two 200-h.p. boilers (new brick settings had to be provided), two feed-water heaters, each 6 ft. in diameter by 16 ft. high, two 100-kw. and one 75-kw. engine generator sets, one 5-panel switchboard and appurtenances, one 400 gal. per min., duplex steam pump, two boiler feed pumps, a hot-water pump, a boiler-room scale, and a rapid sand-filter plant of the wooden-tub type; also, such minor equipment and piping as could be used to advantage. While the equipment was being moved, the institution had to be kept running, and at no time was there any interruption in the service of supplying light, heat, and water to the buildings. It was necessary, however, for the City to furnish and install in the new power house some new equipment and material. This consisted mainly of two 200-h.p. boilers and a 400 gal. per min. duplex

steam pump for pumping filtered water, as well as some new piping, etc. In connection with the new power house, the City also built a reinforced concrete coal-pocket opening direct from the boiler room.

After the new power house was in regular service, the old power house was abandoned, the building wrecked, and the chimney dynamited.

Water Supply.—The institution had two sources of water supply, one, from a sulfur-water well for drinking purposes, and the other, filtered river water for general use. With the filling of the reservoir, the sulfur-water well was flooded, so the City provided a new drinking-water supply by drilling a well located inside the new power house. The river water supply was not so simple. With the old power house, located near the river, the raw-water pumps could take water direct from the river without an excessive suction lift, but with the new power house, at a much higher elevation and at a distance of about 550 ft. from the river, it was too expensive to build a suction well about 60 ft. deep and run a low-level suction conduit to the river so that water could be obtained with the reservoir empty, a condition which would be the same as if no reservoir existed.

The solution adopted for supplying raw water was to build an electrically operated raw-water pumping station out in the reservoir and to run duplicate cast-iron force mains from this station to the power house. The substructure of the pumping station was obtained by extending, down stream, one of the concrete piers of the new Rathbone Bridge, the pier being one located in the old river bed, and constructing in it a suction well and two dry pump wells, each pump well being equipped with a horizontal electrically operated centrifugal pump, automatically and manually controlled from the power house, 700 ft. away. Each pump has a capacity of 500 gal. per min. against a normal head of 42 ft. and a maximum head of 90 ft.

The superstructure of the pumping station is a small brick building, with the floor at practically the same elevation as the floor of the bridge. Access to the building is direct from the bridge floor. At the power house the City built a small reinforced concrete coagulating basin, and also a small filtered-water reservoir of similar construction.

Laundry.—The laundry for the institution was built on somewhat higher ground than the power house, but still not sufficiently high to have escaped partial submergence with the reservoir full. The building was of brick and consisted of three wings, connected by narrow galleries, and had a total floor space of 15 820 sq. ft. Below, was a good basement.

Here, again, it was a case of constructing a new building in another and more convenient location. The new building is exactly similar in construction to the old building, except that one wing was omitted, thereby reducing the floor space to 10 540 sq. ft. In lieu of building the third wing, the City did certain additional work at the new power plant, which the State desired, and other than that which the City would have had to do simply to supply the State with a new power plant equal to the old one.

All the laundry equipment was moved from the old building to the new, the larger fixed pieces consisting mainly of four washing machines, three wringers, large dryer, starch machine, water heater, dry tumbler, large steam

mangle, 26 ironing boards, and 21 electric irons and stands. Including those which were portable, a total of 139 pieces of equipment were moved. Further, all the radiators and steam piping, electric and plumbing fixtures, were moved to and installed in the new building. At no time was laundry work entirely shut down, partial operation having been maintained either in the old building or in the new.

After the new laundry was in service the State took down the old building and salvaged much of the material, with which it constructed another building, somewhat smaller, however, than the old laundry.

Sewage Treatment Works.—Prior to the construction of the reservoir the sewage from the Girls Industrial School flowed by gravity to and through a sewage treatment works, consisting of a screen chamber, a flat-bottom, rectangular septic tank, trickling filters, and intermittent sand filters, the effluent being discharged directly into the river. These works were at such an elevation as to be flooded by the reservoir, so new treatment works, at a higher elevation, had to be built to replace the old ones.

The new treatment works consist of a screen chamber, pumping station, septic tanks, sludge-drying beds, a dosing chamber, and intermittent sand filters, constructed in accordance with plans approved by the Ohio Department of Health. The effluent is discharged into the reservoir about 3 ft. below full reservoir level. The works were designed to treat the sewage from a population of 700.

The screen chamber is a small rectangular structure, 3 by 5 ft. in plan, and is fitted with a bar screen, the bars being spaced 1 in. apart in the clear. From the screen chamber the sewage flows to an automatically controlled pumping station of the pneumatic ejector type. The ejectors are in duplicate, of the Shone make, each ejector having a capacity of discharging 150 gal. at each operation, against a total head of 20 ft., and at least once a minute. The air compressors, of which there are two, are horizontal machines, 6 by 5 in., and of the double-acting, single-cylinder, crank and flywheel type, each compressor being driven by a 5-h.p., 220-volt, direct-current, 1 200 rev. per min., electric motor. The air receiver is 42 in. in diameter and 96 in. high. The equipment is housed in a small brick building, the ejectors being in the substructure.

From the pumping station the screened sewage is pumped to a battery of three hopper-bottom septic tanks. Each tank is 12 ft. square in plan and 13.5 ft. deep from the flow line to the bottom of the hopper. Sewage is admitted at one side of the tank, at the top, and is taken out at the opposite side. Scumboards are provided in front of both the inlet and outlet weirs. Running to the bottom of each tank is an 8-in., cast-iron, sludge discharge pipe so that sludge can be withdrawn without unwatering the tanks.

Septic tanks were chosen for simplicity in operation. The treatment works are in the country, at a sufficient distance from the nearest buildings of the school, as well as being screened from these buildings by a heavy thicket of large trees, so that the question of possible odors could be neglected. The tanks were planned to be operated in rotation, that is, in so far as the digestion of sludge is concerned. When the sludge storage space at the bottom of

one of the tanks is filled, the tank is cut out of commission and allowed to stand idle for a few months, until the sludge is digested. The digested sludge is then withdrawn and the tank put back in service. A second tank is then removed from service and allowed to stand idle, and, finally, the third tank, and so on in rotation. In other words, two tanks are sufficient for sedimentation and sludge accumulation, the third tank serving as a sludge digestion tank.

From the septic tanks sludge is first drawn into a sludge manhole and then discharged from it to the sludge-drying beds. These are three in number, two are 30 ft. square and the third, 10 ft. wide by 30 ft. long. The total sludge-drying area is 2 100 sq. ft. The beds are constructed of 9 in. of sand, supported by screened gravel, and are underdrained with 6-in. tile pipes. The sludge is confined to the beds by pre-cast concrete boards and posts which form the sides and partitions of the beds.

From the septic tanks the settled effluent flows to and through a dosing chamber and thence to the intermittent sand filters. The dosing chamber is 12 ft. wide by 18 ft. long and 3.5 ft. deep, is covered, and is fitted with four automatic timed siphons for dosing the filters, each siphon being piped so as to discharge on to one of two filters.

The sand filters, of which there are eight, are each 69 ft. square, the total sand area being 38 088 sq. ft. The banks surrounding the filter area and subdividing it into the eight beds are of earth construction, seeded and sodded. The filter sand is 30 in. in depth, the effective size being between 0.3 and 0.45 mm. and the uniformity coefficient not exceeding 2, and is supported on a bed of clean gravel in which 6-in. tile under-drains are laid.

After the new treatment works were in operation, the old works were abandoned and material from them was used by the State in miscellaneous construction on the grounds of the institution.

Acquisition of Land

There were three distinct steps in acquiring the land for the O'Shaughnessy Reservoir. They were, first, the determination of the amount of land to be taken from every riparian property affected by the reservoir project; second, the securing of a fair appraisement of the land and improvements to be taken in fee, of flood rights, and of the damage to owners resulting therefrom; and, third, the purchase of the various tracts by negotiation or condemnation, the appraisement being used as a guide for negotiation.

The elevation of the crest of the spillway, plus the maximum depth of water which it was assumed might pass over the same, plus a reasonable additional unflooded strip of land for protection and control fixed, in a general way, the elevation of the taking lines on riparian properties. The city purchased the records of the Franklin County Conservancy District, which consisted of all the data collected in connection with the study of flood relief for the Scioto River Valley, following the great flood in March, 1913, and the topographic maps of that section of the Valley embracing the proposed reservoir site were used in a tentative determination of taking lines on the various properties affected.

Abstracts of the titles of all riparian properties were secured and surveys made of the properties and of the tentative taking lines. The final taking lines were then established, taking into account the relocation of roads on the east side of the reservoir and also the requests of various land owners when the interests of the City were not prejudiced thereby. Complete property maps were then made, which showed for each property the total acreage, the acreage to be taken, and the location of the improvements on the property. These maps were turned over to the City Attorney to whom the purchase of the property had been delegated.

The first step taken by the City Attorney was the selection of a competent Board of Appraisers. The City of Columbus is in Franklin County and the O'Shaughnessy Dam and Reservoir in Delaware County. Five appraisers were selected, four from Delaware County and one from Franklin County. The importance of having a competent appraisement made is reflected in the fact that in the purchase of the 56 tracts acquired by the City, 55 were acquired by negotiation at close to the appraised value, the other being taken to Court where a jury fixed the price within 1% of the appraisement.

The appointment of the appraisers and the fixing of their compensation was authorized by an ordinance of the City Council passed January 9, 1922. The first tract of land was purchased on May 15, 1922, and the last tract on September 23, 1925. The maximum price paid for land was $175, and the minimum, $60 per acre. The land purchases were made by Messrs. Charles A. Leach, City Attorney, John L. Davies, First Assistant, and E. W. McCormick, Second Assistant. A summary of the cost of the reservoir lands, subdivided under various headings, is given in Table 3.

TABLE 3.—Cost of Reservoir Lands.

Items.	Unit.	Number of units.	Total amount paid.	Average price paid per unit.	Average price paid per acre, based on 1 155.9 acres.
Land	Acres	1 155.9	$155 026.75	$ 134.12	$134.12
Flood rights	"	51.7	1 797.00	34.72	1.55
Dwellings	Each	17	59 500.00	3 500.00	51.50
Barns	"	17	16 005.00	941.47	13.85
Other buildings	"	44	8 599.50	195.44	7.45
Springs or wells	"	28	12 050.00	430.36	10.40
Standing timber	.....	.....	5 064.00	.....	4.38
Damage to residue	.....	.....	18 580.00	.....	16.05
Other considerations	.....	.....	5 022.29	.....	4.36
Total to owners	.....	.....	$281 644.54	.....	$243.66
Fees, taxes, assessments etc.	.....	.....	8 488.56	.....	7.34
Total cost	.....	.....	$290 130.10	.....	$251.00

Cost of Work

The total cost of the O'Shaughnessy Dam and Reservoir, exclusive of engineering, was $2 220 080, and a distribution of this cost, subdivided under a number of headings, is given in Table 4. No attempt has been made to dis-

tribute the cost of engineering over the several parts of the work, as it is believed that the costs as presented are of greater value than if they included a percentage for engineering, which, at best, would have to be only approximate.

TABLE 4.—COST OF O'SHAUGHNESSY DAM AND RESERVOIR.

Land, 1 156 acres			$ 290 130
Dam:			
East approach, 483 lin. ft.		$ 29 790	
West approach, 403 lin. ft.		13 580	
Spillway, 879 lin. ft.		635 980	
East bridge abutment, 63.2 lin. ft.		18 440	
West bridge abutment, 63.2 lin. ft.		15 670	
Bridge over spillway, 879 lin. ft.		131 580	
Pavilion:			
Substructure	$ 4 920		
Superstructure	7 940	12 860	
Outlet Gate-House:			
Substructure	$ 64 130		
Superstructure	12 800		
Pipe, gates, and equipment	24 020	100 950	
Power Gate-House:			
Substructure	$ 21 500		
Superstructure	11 470	32 970	
Approach channel to power gate-house		2 700	
Head-race below power gate-house		9 690	
Spillway channels		107 080	
Grouting		15 880	
Engineers' office		9 470	
Unclassified		10 480	1 146 970
Reservoir:			
Clearing and grubbing, 741 acres		$136 320	
Relocation of roads, 7.5 miles		233 050	
Reconstruction of Bridges:			
Rathbone Bridge, 1 046 lin. ft.	$100 110		
Eversole Run Bridge, 40 lin. ft.	5 180	105 290	
Moving church and cemetery		11 130	
Reconstruction at Girls Industrial School:			
Power house	$ 83 850		
Water supply	28 810		
Laundry	40 110		
Sewage treatment works	50 180		
Conduits, sewers, and miscellaneous	89 930	292 880	
Unclassified		4 310	782 980
Total cost, exclusive of engineering			$2 220 080

On the basis of a total available storage capacity of 5 341 000 000 gal., the dam and reservoir, including the land, cost $416 per 1 000 000 gal. available capacity. In Table 5, the cost is subdivided under the headings of land, dam, and reservoir; similar costs are also given, based on the total capacity of the reservoir. The difference in cost per 1 000 000 gal., between that based on available capacity and on total capacity, is just about 1 per cent.

Due to the fact that other engineering work was being designed and constructed under the supervision of the engineering organization that had charge of the design and construction of the dam and reservoir, and at the same time, it would be exceedingly difficult to distribute accurately the cost of engineering to the dam and reservoir and to this other work, notwithstanding that each assistant was required to keep a record of his time. The

difficulty is in distributing accurately the cost of supplies and expenses, and also that part of the pay-roll covering general charge and supervision of the work.

TABLE 5.—COST PER 1 000 000 GALLONS OF O'SHAUGHNESSY DAM AND RESERVOIR.

Items.	Total cost.	COST PER 1 000 000 GALLONS.	
		Total capacity, 5 400 000 000 gal.	Available capacity, 5 341 000 000 gal.
Land	$ 290 130	$ 54	$ 54
Dam	1 146 970	212	215
Reservoir	782 980	145	147
Total, exclusive of engineering	$2 220 080	$411	$416

It is possible, however, to give the total cost of engineering on the dam and reservoir and the other work referred to, which consisted mainly of the enlargement of the water-purification works and the laying of additional cast-iron pipe lines; and it may be said that all this other work would be classed under the general head of water-works construction. The total cost of engineering was $324 810, and, based on a total cost of construction of $3 217 440, the percentage for engineering amounted to 10.10. A distribution of the cost of engineering under the headings of borings and sub-surface investigations, pay-roll, supplies, and expenses is given in Table 6.

TABLE 6.—COST OF ENGINEERING.

Items.	Cost.	Percentage, based on total cost of construction of $3 217 440.
Construction:		
O'Shaughnessy Dam and Reservoir	$2 220 080	69.00
Enlargement of water-purification works	397 140	12.34
Cast-iron force mains	560 920	17.44
Cast-iron distribution mains	34 050	1.06
Miscellaneous	5 250	0.16
Total cost of construction	$3 217 440	100.00
Engineering:		
Borings and sub-surface investigations	$ 10 800	0.34
Pay-roll	271 280	8.43
Supplies	11 110	0.35
Expenses	31 620	0.98
Total cost of engineering	$ 324 810	10.10
Summary:		
Construction	$3 217 440	100.00
Engineering	324 810	10.10
Total cost	$3 542 250	110.10

Beginning and Completion of Work

As stated at the beginning of this paper, the construction of an additional dam and reservoir to provide enlarged storage of raw water was recommended by one of the writers in a report dated August 2, 1919. At a general election held on November 4, 1919, the issuance of bonds in the sum of $3 000 000 for this improvement, together with the extension and enlargement of other parts of the water-supply system, was approved by popular vote.

On March 15, 1920, the City Council passed an ordinance which provided for the issue and sale of these bonds, and on April 12, 1920, an ordinance was passed which provided for the establishment of the Bureau of Water-Works Extension in the Division of Water for the carrying out of these improvements.

The selection of the site for the new dam was made after twenty-six prospective sites between the Village of Dublin and the Powell Road Bridge had been studied. The contract for "The O'Shaughnessy Dam and Appurtenant Works" was awarded on June 6, 1922, to the Thompson-Starrett Company, of New York, N. Y., at an estimated cost of $1 165 511. About two weeks after the signing of the contract, the clearing and grubbing of the site and the erection of the contractor's plant were started. The first concrete was poured on November 1, 1922, and the dam and bridge were completed and ready for use on June 1, 1925.

Construction of the reservoir was carried on coincident with the construction of the dam, and all the work was practically completed by October 1, 1925.

Summary of Principal Data Relating to the O'Shaughnessy Dam and Reservoir

For convenience of reference the principal data relating to the dam and reservoir may be summarized, as follows:

Date of construction	1922-25
Total length of dam, in feet	1 750
Length of masonry part of dam, in feet	879
Length of earth approaches, including concrete abutments, in feet	871
Height of spillway above low water in river, in feet	64
Height of dam from rock foundation to crest of spillway, in feet	84
Height of dam from rock foundation to roadway level, in feet	105.25
Height of roadway above spillway, in feet	21.25
Maximum width of dam at base, in feet	110
Width of paved roadway, in feet	17.5
Width of sidewalks, in feet	4
Net length of spillway, in feet	765.7
Maximum flood capacity of spillway, in cubic feet per second	110 000
Depth of water over spillway with maximum flood, in feet	12.5
Total available storage capacity, reservoir full, in gallons	5 341 000 000

Average depth of water, reservoir full, in feet	19.8
Maximum depth of water, at dam, reservoir full, in feet	69
Length of reservoir, in miles	8
Average width of water surface, reservoir full, in feet	855
Area of water surface, reservoir full, in acres	829
Area of land cleared and grubbed, in acres	741
Area of land purchased, in acres	1 156
Total cost of dam and reservoir, exclusive of engineering	$2 220 080

Contractors

The dam and reservoir were constructed mainly by contract work, the principal contracts and names of the contractors being as follows:

O'Shaughnessy Dam	Thompson-Starrett Company
Diamond drill core borings	Giles Drilling Company
Reconstruction at Girls Industrial School:	
Replacement of buildings	Middle States Construction Company
Moving and replacing equipment and miscellaneous structures	Pitt Construction Company
Replacement of boilers	Heine Boiler Company
Replacement of pumping equipment	Factory Engineering and Sales Company
Centrifugal pumps	Elcon Engineering Company
Relocation of Roads:	
O'Shaughnessy Dam to Rathbone	Uncapher and Gillespie
Rathbone to Bellepoint	J. S. McCann
Reconstruction of Bridges:	
Rathbone Bridge	E. M. Scully
Eversole Run Bridge	Uncapher, Wooley, and Gillespie
Clearing and Grubbing:	
Northern and southeast portions	W. D. Winchell and Company
Southwest portion	J. Livingston and Company

In the construction of the dam, the Thompson-Starrett Company was represented by N. M. Loney, M. Am. Soc. C. E., and in the moving and replacing of equipment and miscellaneous structures at the Girls Industrial School, the Pitt Construction Company was represented by E. D. Harshbarger, Assoc. M. Am. Soc. C. E., President. In the clearing and grubbing, J. Livingston and Company was represented by L. I. Fletcher, Affiliate, Am. Soc. C. E., Resident Manager.

Legislative Procedure

In making the improvements described in this paper it was necessary to comply with State Statutes, with the City Charter, and with City Ordinances, all of which specify terms and conditions governing the expenditure of public

funds. The Legislative Acts required to legalize the carrying out of the work and to both provide and disburse the necessary funds were, as follows:

Statute by State Legislature:		
To authorize agreement with the City to flood State property	1	1
Vote of Citizens at General Election:		
To approve the issue and sale of bonds	1	1
Ordinances by City Council:		
To authorize the issue and sale of bonds	6	
To provide for the engineering organization	9	
To provide for the purchase of supplies and equipment	5	
To authorize contract work	29	
To authorize bed-rock surveys	4	
To authorize the acquirement of property	41	
To authorize miscellaneous projects	22	116
Resolutions by City Council:		
To name the dam	1	
To submit plans to the State Department of Health	2	
To advertise contract work	1	
To appropriate property by condemnation	3	
To modify contracts	5	12
Total Legislative Acts		130

Effect of Additional Storage on Softening of the Water

The O'Shaughnessy Dam was completed June 1, 1925. From May 1 to October 31, 1925, the average daily flow of the Scioto River was 36 000 000 gal. and the average daily water demand of the city, 27 500 000 gal. The flow of the river during 1925 was below normal and would have been a good year for showing the effect of storage in reducing the hardness of the raw water. However, the dam was not completed in time to permit the storage of the soft flood waters of the late winter and early spring months. During 1926 the average daily flow of the river from May 1 to October 31 was 630 000 000 gal., and the average daily demand of the city for the same period was 29 000 000 gal. Consequently, 1926 was not a good year to show the effect of storage, but it is the only one for which data are available (1927).

At the upper end of the O'Shaughnessy Reservoir the influent is made up of the flow from Mill Creek, a tributary having a water-shed of 186 sq. miles, and from the Upper Scioto, which has a water-shed of 770 sq. miles above the confluence with Mill Creek. The water-shed tributary to the reservoir is, therefore, 956 sq. miles; and for further reference it should be stated that the Scioto River has a water-shed of 988 sq. miles at the O'Shaughnessy Dam, 1 052 sq. miles at the Julian Griggs Dam, and 1 076 sq. miles at the water purification works.

Daily samples of water, collected from Mill Creek and the Upper Scioto, were analyzed for mineral content, and the average values weighted on the basis of the contributing water-sheds. The average total hardness of the raw-

water inflow into the O'Shaughnessy Reservoir, from May 1 to September 10, 1926, was 313 parts per million, and the average hardness at the intake to the water purification works, about 22 miles down stream, and with the two reservoirs intervening, which were full all the time and have a total combined storage capacity of 7 120 000 000 gal., was 278 parts per million, a reduction of 35 parts per million in total hardness. During the same period the raw-water draft for the City was 3 911 000 000 gal., and for 1926 the average cost of chemicals for removing 1 part of hardness from 1 000 000 gal. of raw water was $0.104. The apparent saving in the cost of chemicals used in water-softening resulting from storage during the period mentioned, therefore, may be said to be $3\,911 \times 35 \times \$0.104 = \$14\,200$.

Engineering Organization

The engineering work was carried out by the Bureau of Water-Works Extension, a temporary bureau created in the Division of Water and Sewage Disposal by ordinance of the City Council. The Bureau began to function on May 1, 1920. Of the three writers, John H. Gregory, M. Am. Soc. C. E., was Consulting Engineer, and C. B. Hoover, M. Am. Soc. C. E., Engineer in Charge, of the Bureau; C. B. Cornell, M. Am. Soc. C. E., was Construction Engineer in charge of the work at the dam and reservoir. Following the death of Mr. Jerry O'Shaughnessy, Superintendent of the Division of Water and Sewage Disposal, Mr. Hoover was appointed Superintendent of the Division on December 1, 1921, and, in addition to his duties as Superintendent, continued to serve as Engineer in Charge of the Bureau of Water-Works Extension until October 1, 1925, when the work of the Bureau was practically completed. For the first two years, A. R. Holbrook, Assoc. M. Am. Soc. C. E., was Office Engineer, and following his resignation he was succeeded by John C. Prior, M. Am. Soc. C. E. The reservoir surveys were in charge of V. C. Norton, Field Engineer. The Hon. James J. Thomas was Mayor of Columbus and W. H. Duffy, Director of Public Service.

DISCUSSION

J. ERNEST CARMAN,* ESQ. (by letter).—The excavation incident to the construction of the O'Shaughnessy Dam exposed a section of rocks 90 ft. thick in the Valley of the Scioto River and offered a good opportunity for a detailed study of the formations encountered.

The bed-rock exposed along the river valley at the site of the dam is the Columbus limestone of Middle Devonian Age. The excavation for the dam, however, exposed the entire thickness of the Columbus limestone, a few feet of the Delaware limestone above, and about 2 ft. of the Monroe dolomite below, the latter belonging to the Lower Monroe, or Bass Island formation, of Silurian Age. The rocks encountered were correlated by zones with the standard section of the Columbus limestone as determined by Dr. Clinton R. Stauffer.†

The section as seen in the excavation is as follows:

	Thickness, in feet.
Delaware Limestone:	
Bluish limestone in layers of 3 to 6 in.....	3+
Columbus Limestone:	
Zone H. Bluish gray, fossiliferous limestone in layers of 1 ft. to 2 ft. The base is at a very smooth plane.....	7
Zone G. Bluish gray, fossiliferous limestone in layers of 3 to 5 ft.....	19
Zone F. Bluish gray, very fossiliferous limestone with *Spirifer gregarius* (*Spirifer gregarius* Zone).....	4
Zone E. Bluish gray limestone that weathers buff. In layers of 12 to 18 in. Upper half fossiliferous.....	14
Zone D. Grayish brown limestone with bands of chert nodules and some chert layers (Chert Zone). Contains some gastropods.....	12½
Zone C. Missing.	
Zone B. Brown dolomitic limestone with some banding and in thick ledges of 3 to 5 ft. The basal 8 ft. is more compact and contains some sand grains.....	30
Zone A. Arenaceous limestone matrix with pebbles of Monroe dolomite and chert (Conglomerate Zone)....	½
Monroe Dolomite:	
Ashen gray to drab, laminated, argillaceous dolomite in beds of 2 to 8 in. In part, brecciated, with minute fragments	2+

Zone A represents the earliest deposit of the Columbus time as the sea transgressed the eroded, pebble-strewn surface of the Monroe dolomite. The sand grains were either on the surface or were washed in by the encroaching sea. Their characteristics are such as to show that they were derived from the Sylvania sandstone of Northern Ohio. The sand content of this layer at this location is greater than at any other known exposure in Central Ohio.

* Prof. of Geology, Ohio State Univ., Columbus, Ohio.
† *Bulletin No. 10*, Geological Survey of Ohio, 1909.

Zones G, E, and B were broken by vertical and inclined joints, a condition having special significance from an engineering viewpoint, for the foundation of a dam.

Zone H forms the upper few feet of the rock wall on either side of the valley, but the top of the zone is not exposed. At a point 50 yd. east of the east end and at a slightly less distance west of the west end of the concrete portion of the dam the "bone bed" which marks the top of the Columbus limestone was encountered in the excavation for the core walls of the earth embankment sections of the dam. Beyond these points the Delaware limestone was encountered in the core-wall trenches.

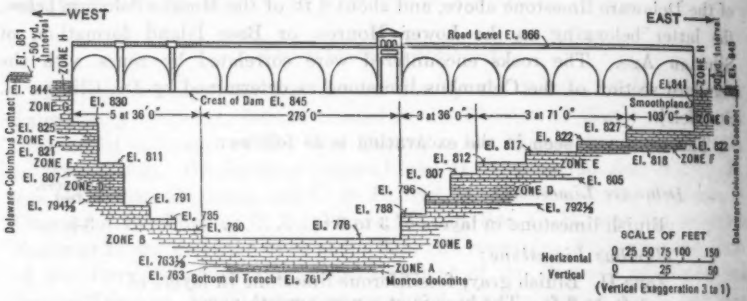

FIG. 25.—CROSS-SECTION OF THE SCIOTO VALLEY AT THE DOWN-STREAM FACE OF THE O'SHAUGHNESSY DAM.

It is of interest to note that the study developed the facts that the Delaware limestone on the east and west sides of the valley is ¼ mile apart and the Delaware-Columbus contact is at about Elevation 850 above sea level; whereas, the geological maps, based on the best information previously available, show a separation between the two areas of Delaware limestone of 1½ miles and a Delaware-Columbus contact about 45 ft. higher than was actually found.

Fig. 25, shows a cross-section of the Scioto Valley at the dam, as one sees it looking up stream (north) from just below the dam. The section shows, to scale, the rock benches just south of the dam and the location of the exposures of the several zones of the Columbus limestone with respect to these benches and to the valley sides.

B. F. JAKOBSEN,* M. AM. SOC. C. E. (by letter).—The paper indicates that the work was carefully planned and executed, as of course it ought to be. The writer notes the absence of any reference to the water-cement ratio in connection with the concrete. It would be interesting to know how the concrete was tested and what strength was found.

Before concreting was started on the Pacoima and Santa Anita Dams of the Los Angeles County Flood Control District, tests on different concretes were made at the Pacoima Testing Laboratory under the writer's direction. At first, hand-mixing was used, but this was found to give such widely varying results, that no conclusions could be drawn. Therefore, a small motor-

* Cons. Engr. (La Rue & Jakobsen), Los Angeles, Calif.

driven mixer was installed, and the results were found to check very closely and consistently with the Abrams curve for strength against water-cement ratio. The results obtained at Pacoima have already been reported.* All the tests showed that the water-cement ratio was by far the most important feature of concrete production and that when this was watched good concrete resulted.

The desirability of strong and durable concrete has been emphasized often enough and there seems also to be a more or less direct relationship between the impermeability of concrete and its strength. In dam construction the writer thinks that shrinkage may be the most important factor, and it seems well established that the leaner the concrete the less will be the shrinkage. Therefore, for equal strength, the leaner concrete is to be preferred, especially in great structures where shrinkage effects may produce high tensile stresses.

Impermeability is also of great importance, especially in cold countries. A Norwegian engineer, Fredrik Vogt, Assoc. M. Am. Soc. C. E., has told the writer, that his experience in Europe indicates that in warm climates porous concrete tightens up with age, whereas in cold climates the reverse is the case. This seems to agree with American experience.†

The main part of the O'Shaughnessy Dam is constructed with Class A concrete (see Fig. 14), but a 12-in. layer of a richer mix (Class C concrete) was added all around the dam. The writer doubts the advisability of adding such a surface coating and is inclined to believe it is detrimental rather than beneficial. This richer mix will shrink more and will probably crack in a great many places, thus tending to produce cracks in the underlying concrete. The writer realizes the desirability of increasing the resistance of the concrete to scour, where it is subject to the action of rapidly moving water, but this ought to be accomplished without increasing the tendency to shrink in setting.

W. H. DITTOE‡ AND G. GALE DIXON,§ MEMBERS, AM. SOC. C. E. (by letter).—For the past twenty years the City of Columbus has presented an outstanding example among the moderately large American cities in its progressive attitude toward public water supply, an example established by the excellent initial development involving softening works completed in 1908, and maintained by further improvements recently finished (1928).

The authors have completed a thorough-going record of the system by adding this paper to the earlier exhaustive description‖ of the initial development.

The architectural effects of the O'Shaughnessy Dam are dignified and appropriate, and evidently the construction work is excellent; the design appears properly conservative in flood-discharging capacity and in the static resistance of the masonry overflow dam.

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* *Transactions*, Am. Soc. C. E., Vol. 90 (1927), p. 583.

† "Corrosion of Concrete," by John R. Baylis, Assoc. M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 90 (1927), p. 791; also, pp. 792 and 816.

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§ Deputy Chf. Engr., The Mahoning Val. San. Dist., Youngstown, Ohio.

‖ *Transactions*, Am. Soc. C. E., Vol. LXVII (June, 1910), p. 206.

The "cascade" treatment of the natural rock slopes of the valley below the dam, for returning the spillway discharge to the channel of the river, is of special interest. Under flood condition similar to that of 1913, the spillway will pass a 9.5-ft. depth of water over its crest, and for about 49% of its length will discharge directly upon the unprotected limestone floor surfaces of the cascade at elevations above tail-water level; this discharge will travel over the cascade floor in thin high-velocity sheets at "the lower alternative stage" of flow. During the discharge of smaller flows, this condition will rule over a still greater width of cascade floor, while for the maximum anticipated flood—a depth of 12.5 ft. over the crest of the spillway—the higher stage of tail-water will result in only a small reduction of width of floor unprotected by any back-water.

The writers have been impressed by the excellent opportunity for a full-scale study of the occurrence of the hydraulic jump and related phenomena bearing on the subject of erosion below over-fall dams, which exists at the Julian Griggs Dam. Considerable experimental work with small-scale models has recently been done along these lines, but there is a very definite need for systematic observations by coincident photographs and measurements of the performance of actual structures. The Julian Griggs Dam is of the ogee type, with a uniform height of 33 ft., and is subject to overflow depths which have ranged to 13 ft. Marked erosion of the limestone river bottom beyond the apron has occurred. At the lower stages, a well-defined "standing wave" appears just at the end of the short apron, which at higher stages is drowned out by the natural back-water of the channel.

It is believed by the writers that a far-sighted policy in the management of domestic water supplies in populous districts requires the acquisition of a more liberal width of marginal land surrounding reservoirs than has apparently been considered necessary at Columbus, and also a rigorous control of the use of land and water surface, to afford positive protection against serious pollution. In recent years a growing tendency has manifested itself, in the vicinity of centers of population, toward the development of summer cottage colonies near all bodies of water within easy reach which, in any measure, might afford recreational opportunities. Unless the responsible authorities take steps calculated definitely to discourage such developments in the vicinity of water supply reservoirs, the value of these reservoirs for their primary purpose may, in time, become seriously impaired.

It is the understanding of the writers that the use of the O'Shaughnessy Reservoir for recreational purposes has already gained headway through an ordinance of the Columbus City Council permitting boating and fishing, and that land immediately abutting the reservoir property has been allotted and is being sold by realty companies for residential purposes.

A striking example of the results which may follow the adoption of a lenient policy in regard to recreational use of the water and the marginal lands of a reservoir is to be found in the Milton Reservoir on the Mahoning River in Northeastern Ohio, built by the City of Youngstown in 1917 in connection with its water supply system. The Milton Reservoir is located about

16 miles west of the city and at a distance of about 40 miles up stream from the Youngstown Water-Works Intake from the Mahoning River within the city. The reservoir was constructed for use in regulating the dry-weather flow of the river, not only for the benefit of the municipal supply, but also for industrial purposes and, incidentally, for preventing nuisance from the discharge of the city's untreated sewage into the stream. The purpose of construction was therefore not parallel with that under discussion, but the result of the policy adopted for control of the reservoir is none the less an outstanding lesson.

The Milton Reservoir is formed by a dam 55 ft. high, flooding 1 657 acres of land and impounding 9 000 000 000 gal. of water. The initial purchases of land by the City comprised 3 344 acres, or about double the flooded area. Much of this property was acquired under deeds which reserved to the grantors the right of access to and the use of the reservoir for boating, fishing, bathing, and other uses normal to an ordinary body of water. Subsequently, the City established a "permanent holding line" at a relatively short distance from the water's edge enclosing about 2 500 acres—about one and one-half times the flooded area—and the "salvage land" lying outside this line was divided into blocks and offered for sale. Under these conditions, an extensive development of the surrounding lands for summer-resort purposes has naturally followed, and the reservoir has become an important recreational center. About twenty-five separate allotments offering summer camp and cottage sites have been put on the market, and restaurants, bath and boat houses, dance halls, and other semi-public buildings have been constructed for the service of a summer population of several thousand people.

TABLE 7.—WIDTH OF MARGINAL LANDS ACQUIRED ABOUT VARIOUS IMPOUNDING RESERVOIRS.

Reservoir.	Land acquired, in acres.	Area flooded, in acres.	Ratio of land acquired to area flooded.	Length of shore line, in miles.	Mean width of marginal strip, in feet.
New York, N. Y.:					
Ashokan	15 222	8 180	1.86	40	1 450
Kensico	4 500	2 218	2.03	30	630
Schoharie	2 372	1 145	2.07	16.5	610
Boston, Mass.:					
Wachusett	5 163	4 200	1.23	35	230
Providence, R. I.:					
Scituate	12 547	3 600	3.5	38	2 150
Akron, Ohio:					
Rockwell	2 065	769	2.7	20	535
Columbus, Ohio:					
Griggs	452	363	1.2	....	65
O'Shaughnessy	1 159	829	1.4	....	250
Youngstown, Ohio:					
Milton	3 344	1 657	2.0	23	606
The Mahoning Valley Sanitary District, Ohio:					
Meander Creek (proposed)	5 248	2 610	2.6	40	690

The writers have recently had occasion to consider the availability of Milton Reservoir for the purpose of providing a permanent source of public

water supply for the Cities of Youngstown and Niles, Ohio, in connection with the project of the Mahoning Valley Sanitary District for providing a complete modern system of works for these cities. The existing conditions about the reservoir are entirely proper in connection with its present function of general regulation of the river, but radical procedure would be required in preparation for its use as a permanent source of potable supply. Careful estimate indicated that in order to gain the necessary sanitary control of the waters and the marginal lands, the taking of an additional area of 3 000 acres of land over and above that now owned by the City would be required, at a cost exceeding $2 000 000. The original total cost of the reservoir, including land and construction, was $1 321 000.

The Mahoning Valley Sanitary District is now (1928) preparing to develop a relatively uncontaminated upland source of supply for Youngstown and Niles, so that these cities will shortly abandon the use of the badly polluted Mahoning River water. In connection with this project, the writers have examined precedent in the acquisition of land about various important reservoirs, and submit Table 7, as of possible interest.

It should be noted that, with the exception of the Columbus Reservoir, the "mean width of marginal strip", as stated in Table 7, is derived by dividing the area of land taken above reservoir level by the recorded length of flow line; irregularity of shore line, of course, has an important bearing on the practical fixing of taking lines which is not evaluated in the arrangement of Table 7. Thus, the proposed Meander Creek Reservoir has a very irregular shore line, whereas the two Columbus reservoirs are of quite smooth outline, each being confined principally to the main valley of the stream.

JOHN H. GREGORY,* C. B. HOOVER,† AND C. B. CORNELL,‡ MEMBERS, AM. SOC. C. E. (by letter).—In his discussion, Mr. Jakobsen mentions three points which merit further explanation, namely, the absence of any reference to the water-cement ratio, in connection with the mixing of the concrete; the methods of testing the concrete and the results of these tests; and the desirability of facing the lean concrete forming the main body of the dam with a richer mixture of concrete. As bearing on the first point, it should be stated again that concrete was first placed in the dam on November 1, 1922. At that time, engineers were generally of the opinion that there was a direct relation between the compressive strength of concrete and the quantity of water used in mixing. It had also been shown in laboratory experiments that the quantity of mixing water necessary to produce a workable concrete had a direct relation to the grading of the aggregates used, the fineness modulus being the means by which this relation was expressed. The application of these principles to large construction work and the development of satisfactory methods of field control were in the experimental stage. Little, if anything, was known concerning the weather-resisting properties of concrete proportioned according to the new theories, nor of its behavior when exposed to alternate wetting and drying, because long-time proof on these points was still lacking.

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‡ Field Engr., Mahoning Val. San. Dist., Youngstown, Ohio.

The engineers in charge of the design and construction of the O'Shaughnessy Dam were confronted with the problem of building a structure in which long life, a high degree of imperviousness, the greatest possible weight per unit of volume, and maximum resistance to the effects of temperature and water must be obtained; and, in which, the strength of the main body of the structure was of relatively minor importance, the maximum compressive stress being less than 100 lb. per sq. in.

At the time construction was started, it appeared to be very generally accepted by engineers that concrete proportioned to secure maximum density fulfilled these requirements, and, therefore, this basis of proportioning the concrete mixtures was adopted. In the prosecution of the work the quantity of mixing water was at all times held to the minimum amount which would secure the workability necessary for the satisfactory placing of the concrete, consideration, of course, being given to the location of the concrete being placed; and, furthermore, concrete was not transported by means of chutes because their use was not permitted by the specifications.

Numerous mechanical analyses of the four sizes of aggregates used were made and a large number of combinations of the results secured were plotted for comparison with the practical curve of maximum density, as established by Taylor and Thompson. Trial batches, containing cement, sand, coarse aggregates, and water, mixed by hand, were made up of the combinations which most nearly approached the Taylor and Thompson curve, and of variations from these combinations in which the total volume of aggregates was kept the same while the relative proportions of the several aggregates were changed. Each trial batch was measured for volume by placing the concrete in a pipe, the workability of the batch was observed and recorded, and, also, cylinders 6 in. in diameter and 12 in. long were made and broken, for a check on the compressive strength. From the results of these tests the various mixtures were selected. Throughout the work only slight variations from these proportions were made; for example, a small increase in the fine aggregate with a corresponding decrease in the coarse aggregates was allowed in thin walls and in the arch rings of the bridge; and, in the latter case, the cement, also, was slightly increased to secure greater workability.

From time to time throughout the course of the work test cylinders were made and broken. Cylinders made from concrete containing the two smaller sizes of coarse aggregate showed greater consistency in the test results than those containing the three sizes. It was demonstrated conclusively, in the case of a few of the tests, that cylinders 6 in. in diameter and 12 in. long were not as reliable as larger cylinders for testing concrete containing the 3-in. aggregate.

In addition to the proportioning and strength tests made by the engineers, J. R. Shank, M. Am. Soc. C. E., was commissioned by the contractor to conduct an extensive series of tests, at the beginning of the work, with various combinations of the aggregates which were to be used. Several hundred test cylinders were made and broken and the results were substantially in accord with those secured by the engineers. Since none of the tests was made to set

up a water-cement ratio for the work, a detailed account of the test results is thought to be out of place in this discussion.

A richer concrete facing (1:2:4), 1 ft. thick, was placed on the exposed faces of the dam and bridge piers for the purpose of securing greater resistance to scour, to alternate wetting and drying, and to changes of temperature. Experience with previous work in Columbus, especially the Julian Griggs Dam, convinced the writers that resistance to scour and to temperature and water exposure would be secured best and at a minimum expense by applying a 1:2:4 concrete on the face of the overflow and on the vertical sides of the abutment sections, as well as on other exposed concrete structures. With the facing used no difficulty has been experienced with shrinkage cracks in the Julian Griggs Dam and, to date (1929), none has appeared in the O'Shaughnessy Dam.

The discussion of Messrs. Dittoe and Dixon raises a number of very interesting points. The most important of these has to do with scour at the toe of the two dams mentioned; that is, the Julian Griggs Dam, which was put under construction early in 1904 and practically completed in December, 1905, and the O'Shaughnessy Dam which was put under construction in 1922 and completed, ready for use, on June 1, 1925. Both structures are built on limestone, the geological formations at the two sites being, in general, similar.

Referring first to the Julian Griggs Dam, records have been kept by the City of Columbus since 1904, which show the scour at the toe of the dam. These records include elevations of the original surface of the ground and the top of the rock before construction started. Cross-sections were again taken on September 1, 1908, October 14, 1909, July 14, 1913, October 26, 1920, and the last ones on August 10, 1928.

Scour continued year after year, from the time the dam was completed in 1905 until 1920, that is, for about fifteen years, but, in the last eight years, since 1920, practically no further scour has occurred. On a few of the sections some further small scour has occurred since 1920, whereas, on the other hand, on some of the other sections there appears to have been a filling up. This is probably due to the movement of some loose rock on the bottom of the river. Taking the sections by and large it seems that a condition of practical stability below the toe of the dam was reached in 1920.

Attention should also be called to the fact that the dam, in March, 1913, passed through the most disastrous flood on record in the history of Central Ohio. At the peak of the flood on March 25, the depth of water passing over the 500-ft. spillway was 12.8 ft., with a flood discharge of approximately 80 000 cu. ft. per sec.

A full set of cross-sections was not taken in 1913, but scour occurred between 1909 and 1913. It is not known, however, whether the scour was progressive or whether it took place mainly at the time of the March, 1913, flood. The deepest scour, as of 1913, was at Stations 8 + 96 and 9 + 52 (see Fig. 26), since that time the river bottom has filled up considerably.

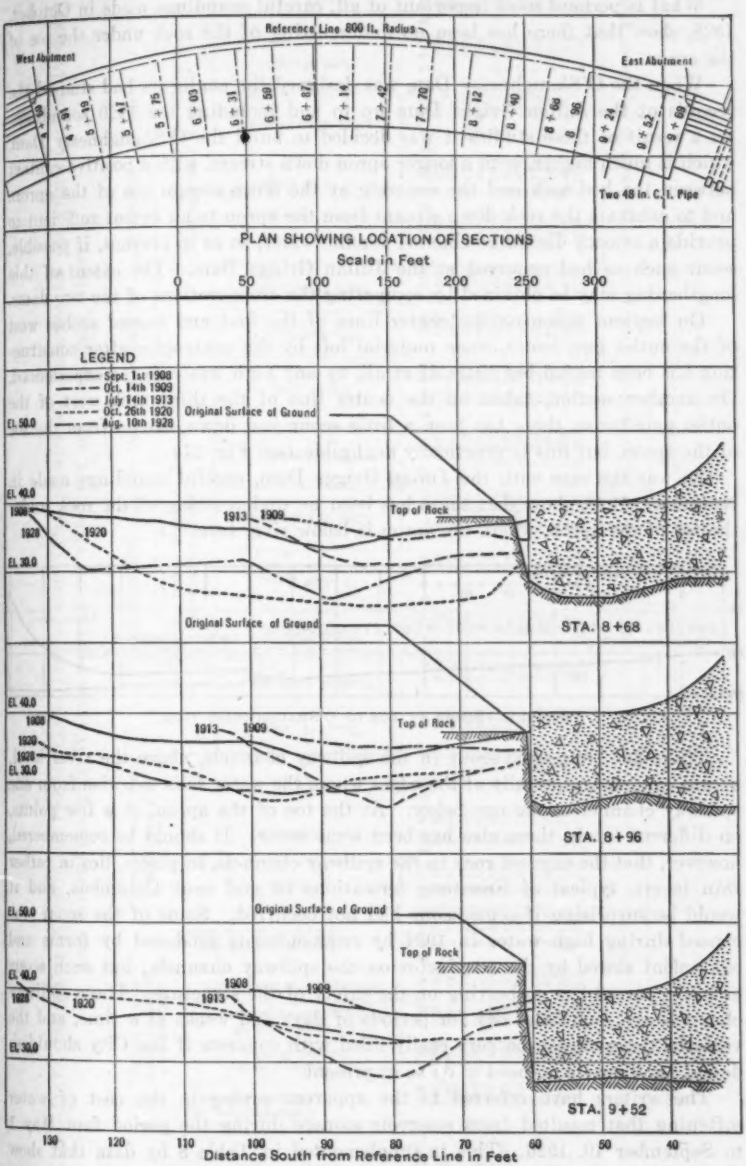

FIG. 26.—SCOUR AT TOE OF JULIAN GRIGGS DAM.

What is perhaps most important of all, careful soundings made in October, 1928, show that there has been no undercutting of the rock under the toe of the apron.

When the O'Shaughnessy Dam was designed the engineers had studied the records of the Julian Griggs Dam up to and including the 1920 soundings. As a result of these studies it was decided to build the O'Shaughnessy Dam, which is much higher, with a longer apron down stream, with a positive contact between the bed-rock and the concrete at the down-stream toe of the apron, and to excavate the rock down stream from the apron to an extent sufficient to provide a smooth discharge channel for the water, so as to prevent, if possible, scour such as had occurred at the Julian Griggs Dam. The extent of this lengthening may be obtained by comparing the cross-sections of the two dams.

On sections taken on the center lines of the first and second arches west of the outlet gate-house, some material left by the contractor after construction has been moved but little, if at all, by any high water so far experienced. On another section, taken on the center line of the third arch west of the outlet gate-house, there has been a little scour just down stream from the toe of the apron, but this is practically negligible (see Fig. 27).

As was the case with the Julian Griggs Dam, careful soundings made in November, 1928, show that there has been no undercutting of the rock under the toe of the apron, where the apron is below river level.

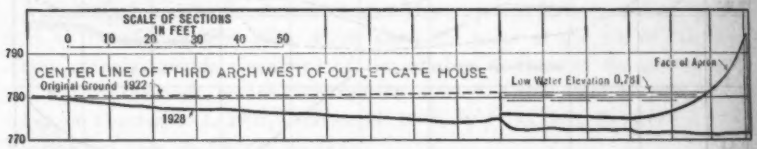

FIG. 27.—SCOUR AT TOE OF O'SHAUGHNESSY DAM.

There has been some scour in the spillway channels, above the river level, as was expected, especially at the edges where the water falls sidewise from one spillway channel to the one below. At the toe of the apron, at a few points, on different levels, there also has been some scour. It should be remembered, however, that the exposed rock in the spillway channels, in places, lies in rather thin layers, typical of limestone formations in and near Columbus, and it would be surprising if some scour had not occurred. Some of the scour was caused during high water in 1924 by cross-currents produced by forms and equipment stored by the contractor on the spillway channels; but such scour as has occurred has no bearing on the safety of the structure. These spillway channels are sometimes dry for periods of days and weeks at a time, and the voids in the rock can be very easily filled with concrete if the City should so desire, but there is no need to do so at present.

The writers have referred to the apparent saving in the cost of water softening that resulted from reservoir storage during the period fom May 1 to September 10, 1926. This is supplemented in Table 8 by data that show the effect of storage in the Julian Griggs and O'Shaughnessy Reservoirs on the cost of purifying and softening the water supply of Columbus for a period of

TABLE 8.—DATA SHOWING EFFECT OF THE JULIAN GRIGGS AND O'SHAUGHNESSY RESERVOIRS ON THE COST OF PURIFYING AND SOFTENING THE WATER SUPPLY OF THE CITY OF COLUMBUS, OHIO.

Month.	Consumption, in million gallons.	Total Hardness, in Parts per Million.			Saving in cost due to decrease in hardness.	Cost of Softening Chemicals per Ton.			Quantity Required and Cost of Coagulating and Softening Chemicals.													
									Actual, for Inflow Water to Water Purification Works.							Computed, for Inflow Water to O'Shaughnessy Reservoir.						
		At Water Purification Works.	At upper end of O'Shaughnessy Reservoir.	Decrease due to storage.		Lime.	Soda ash.	Alum.	Tons of lime.	Tons of soda ash.	Tons of alum.	Cost of lime.	Cost of soda ash.	Cost of alum.	Total cost.	Tons of lime.	Tons of soda ash	Tons of alum.	Cost of lime.	Cost of soda ash.	Cost of alum.	Total cost.
July, 1927.	959	247	282	35	$ 2 694	$6.69	$28.60	$12.74	657	164	77	$ 4 395	$ 4 690	$ 981	$ 10 066	781	315	137	$ 5 225	$ 9 009	$ 1 745	$ 15 979
August, "	915	244	338	94	6 903	6.69	28.60	12.74	615	216	74	4 114	6 178	943	11 235	884	442	65	5 914	12 641	828	19 383
September, "	930	261	330	69	5 150	6.69	28.60	12.74	637	259	70	4 262	7 407	892	12 561	860	453	66	5 753	12 956	841	19 550
October, "	890	280	352	72	5 165	6.69	28.60	12.74	646	294	74	4 322	8 408	943	13 673	920	460	64	6 155	13 156	815	20 126
November, "	920	299	371	72	4 738	6.69	28.60	12.74	680	305	84	4 549	8 723	1 070	14 342	920	532	58	5 486	15 215	739	21 440
December, "	865	148	282	134	9 303	6.69	28.60	12.74	377	179	314	2 522	5 119	4 000	11 641	638	338	124	4 268	9 667	1 580	15 515
January, 1928.	933	250	270	20	1 498	6.57	27.74	11.50	674	293	147	4 428	8 128	1 690	14 246	695	310	133	4 566	8 599	1 530	14 695
February, "	830	192	246	54	3 597	6.57	27.74	11.50	516	207	268	3 390	5 742	3 082	12 214	556	292	178	3 653	8 100	2 047	13 800
March, "	856	237	268	31	2 130	6.57	27.74	11.50	593	226	119	3 896	6 269	1 368	11 533	627	335	184	4 119	9 293	2 116	15 528
April, "	816	189	255	66	4 322	6.57	27.74	11.50	484	210	299	3 180	5 825	3 338	12 343	575	247	117	3 778	6 852	1 345	11 975
May, "	927	222	358	136	10 118	6.57	27.74	11.50	656	159	89	4 310	4 411	1 023	9 744	968	440	66	6 360	12 206	759	19 325
June, "	878	206	273	67	4 721	6.57	27.74	11.50	588	207	227	3 863	5 742	2 610	12 215	696	330	188	4 573	9 154	2 162	15 889
Total.....	10 623	2 775	3 625	....	$60 339	.....	.....	.....	7 123	2 719	1 842	$47 231	$76 642	$21 940	$145 813	9 020	4 494	1 880	$59 850	$126 848	$16 507	$203 205
Average....	887	231	302	71	$ 5 028	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....

one full year from July 1, 1927, to July 1, 1928. Samples of water were collected daily from Mill Creek and the Upper Scioto, and at the end of each month composite samples of each were analyzed and the average analysis of the inflow water at the upper end of the O'Shaughnessy Reservoir was then taken as that secured by weighting the separate analyses on the basis of the areas of the contributing water-sheds.

The tons of lime, soda ash, and alum actually used at the Water Purification Works each month, together with the cost of each chemical and the total chemical cost, are given in Table 8. In this table also are given the computed tons of chemicals which the inflow water would have required, and the corresponding chemical cost if the inflow water had been treated so as to produce a water of the same quality as that delivered by the Water Purification Works.

The method of computation used in determining the tons of lime, soda ash, and alum required for the inflow water, when applied to the average monthly composition of the water actually treated, gives chemical tonnage values of about 4% less than those actually used in the treatment of the water, so that the method of computation, as applied to the inflow water, gives a conservative basis for comparison.

The tabulated data show that the annual saving in cost of the three chemicals required, due to reservoir storage, was ($203 205 — $145 813 =) $57 392, and that the saving in cost of lime and soda ash alone was ($186 698 — $123 873 =) $62 825. The difference in cost is due to the increase in the quantity of alum required for the treatment of the water because of the persistence of turbidity for longer periods of time than would have been the case in the absence of reservoir storage. The average cost of lime and soda ash to remove 1 part of total hardness from 1 000 000 gal. for the year was 8 cents. The saving in cost was, therefore, 10 623 000 000 gal. $\times$ 71 parts of total hardness $\times$ $0.08 = $60 339.

The total saving of $57 392 in the cost of chemicals for the period of one year from July 1, 1927, to July 1, 1928, is equal to the annual interest on an investment of approximately $1 150 000 at 5%; also, it represents a reduction of 28% in the cost of softening and purifying chemicals.

More than three years have now (1929) passed since the dam was completed, and a careful inspection of the structure fails to reveal any defects or any visible leakage around or through the dam.

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Paper No. 1716

THE TREATMENT OF THE WATER SUPPLY OF THE CITY OF COLUMBUS, OHIO*

By Charles P. Hoover,† Esq.

With Discussion by Messrs. F. H. Waring, George W. Fuller, Robert Spurr Weston, and C. W. Foulk.

Synopsis

A description of construction features of the Water Softening and Purification Works as originally built at Columbus, Ohio, will not be given in this paper. This has been done‡ by John H. Gregory, M. Am. Soc. C. E., who was Engineer in charge of the design of the plant. Operation and new features only will be discussed.

Original Water Supply

Prior to the time the Purification Plant was built, the water supply of Columbus was not safe for drinking. It was badly polluted with sewage, always turbid, and, following heavy rains, extremely muddy. The turbidity of the Scioto River averages 75 parts per million. During floods this rises to 2 250 parts per million, meaning that a day's supply carries from 200 to 300 tons of mud in suspension.

The bacterial count (37° cent., 24 hours) averages 5 665 per cu. cm. The *B. Coli* count sometimes reaches 100 000 per 100 cu. cm. In 1908, the typhoid fever death rate reached 138 per 100 000 population. During the summer of that year there were 1 500 cases of typhoid.

The original supply of water was unsatisfactory for domestic or industrial purposes because it was highly mineralized. The average hardness of the

* Presented at the Fall Meeting of the Society at Columbus, Ohio, October 12, 1927.

† Chemist in Chg., Water Softening and Purification Works, Columbus, Ohio.

‡ "The Improved Water and Sewage Works of Columbus, Ohio," by John H. Gregory, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXVII (1910), p. 206.

present raw water from 1909 to 1926, inclusive, was 272 parts per million, with a maximum of 495 parts per million. This hardness is due to calcium carbonate, magnesium carbonate, calcium sulfate and magnesium sulfate. There are 68 570 lb., or more than a carload, of these compounds in a day's supply. There is enough calcium to make 48 tons of hydrated lime and the magnesium in one day's supply would make 62 500-qt. bottles of milk of magnesia.

Calcium and magnesium salts are very objectionable. They form deposits in boiler tubes and hot-water heating systems, resulting in large heat losses. They also re-act with soap to form slimy, insoluble compounds, which can be seen in the water before lather begins to form. Soap has no cleansing action until it lathers, therefore, much of it is wasted in hard water.

Before the water supply of the city was softened, cistern water was used almost generally for domestic purposes. More than 17 000 cisterns were in use in Columbus, when the municipal plant was built. Cisterns are only makeshifts for a suitable municipal water supply. The water in them is hardly ever adequate, and is almost always dirty and, sometimes, not even soft. They have now been practically abandoned in Columbus.

Water from individual private wells was used for drinking purposes, but many of these wells were contaminated with sewage. They were condemned by the Board of Health after the Purification Plant was put into service.

Status of Water Filtration and Water Softening in 1904

At the time (1904) the Columbus plant was being designed, only two large mechanical water filtration plants of concrete construction were in operation, one at Little Falls, and one at New Milford, N. J. Valuable and helpful experimental work on water purification had been done by George W. Fuller, M. Am. Soc. C. E., who was one of the consulting engineers on the design of the plants, at Louisville, Ky., and Cincinnati, Ohio, on the treatment and filtration of water,* but no large combined water-softening and filtration plant had been built. There was little or no precedent to follow. Under the supervision of George A. Johnson, M. Am. Soc. C. E., bottle and barrel water-softening experiments were made at the Columbus Sewage Testing Station. The results of these experiments were used as a guide in the design of the plant. The demand for the new plant was so urgent that there was no time to investigate the Columbus problem extensively, as was done at Louisville and Cincinnati.

Flexible Plant Built

The plant, as built, was provided with facilities for softening and filtration. The design was so elastic that the operating process could be modified without making elaborate changes. Numerous places for applying the chemicals were provided. The water could be separated and treated in portions, also the time of mixing could be varied. The mixing and settling basins could be operated in series, or parallel, or in series parallel. Raw water could be applied at many points to correct for over-treatment.

* Report of Water Purification at Louisville, Ky.; and Report of Water Purification at Cincinnati, Ohio.

As a result of this flexibility, after twenty years of continuous use, the plant is being operated (1927) as effectively as the most recently built plants. No extensive or costly additions and changes have been required to accomplish this; it has never been necessary to shut down for repairs, changes, or enlargements. The plant has been shut down completely only once since it was put in operation in 1908. This was necessary during the 1913 flood, when one of the force mains, laid in the river bed, was washed out, making it impossible to maintain pressure in the distributing system, and this shut down lasted for only eight hours.

Character of Raw Water Changes Suddenly

The design of the plant was also complicated because the Scioto River, from which Columbus takes its supply, is very flashy. It is sometimes clear, sometimes muddy, sometimes soft, and sometimes hard; and as the waterworks is not provided with large settling basins, in which to store raw water to equalize these fluctuations, the plant must take whatever is flowing in the river. Table 1 shows the fluctuations in the raw-water analysis during a 48-hour period from March 21 to 23, 1916.

TABLE 1.—Character of Scioto River Water.

Date.	Turbidity.	Carbonate hardness.	Non-carbonate hardness.	Total hardness.
March 21.....	22	189	96	285
March 22.....	678	133	63	196
March 23.....	2 250	46	18	64

It was known by those who designed the plant that these sudden changes in the analysis of the raw water would occur. Specially designed and readily adjustable devices, capable of feeding small and large quantities of chemicals, were provided.

These sudden changes require constant supervision to treat the water properly. A chemist is kept on duty day and night making tests once every hour. At plants where only coagulation and filtration are practiced a change in the character of the raw water coming to the plant does not present as serious a problem to the operator as it does at a water-softening plant. In the former it is only necessary to increase the application of the coagulating re-agent; whereas, at a water-softening plant, it is not only necessary to increase the application of the coagulating re-agent, but also to diminish the quantity of softening chemicals.

The addition of too much lime gives the water a caustic reaction. The addition of too small a quantity of lime may prevent the coagulant from completely precipitating, resulting in the passage of alum floc through the filters and the production of a turbid effluent. Lime, unless added in exactly the right quantity, may be detrimental to successful purification and clarification processes.

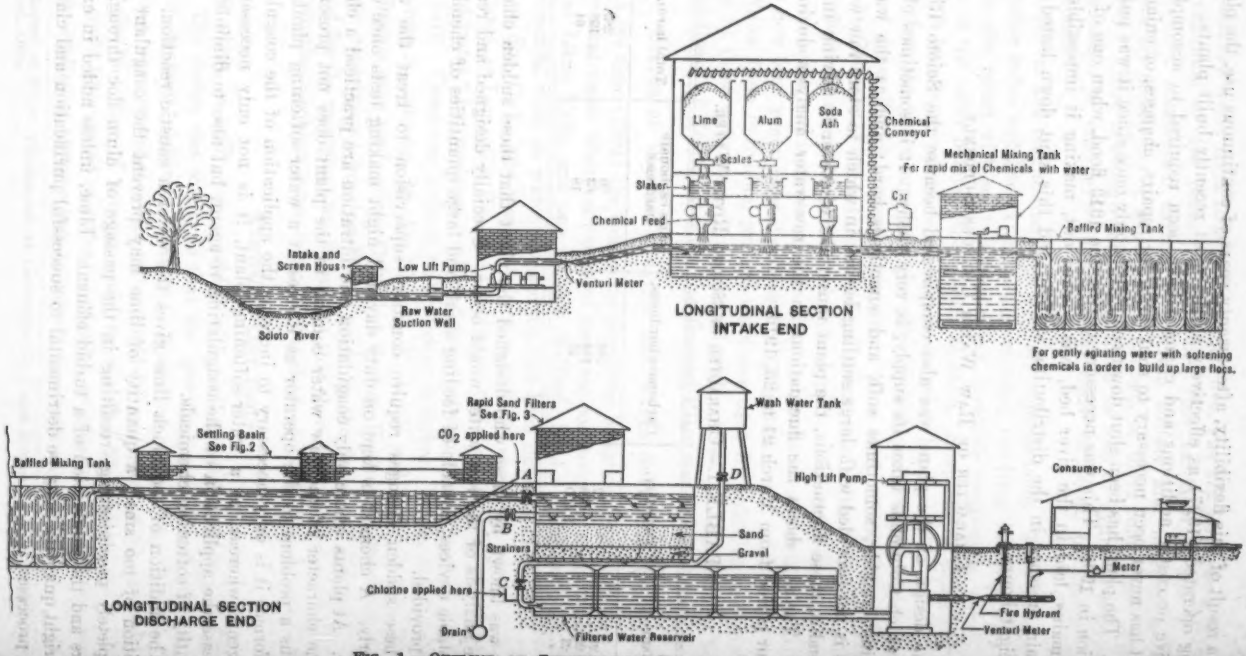

FIG. 1.—OUTLINE OF PURIFICATION PROCESS, COLUMBUS, OHIO.

Process Used in Treatment of Water Supply

The water flows, from the river (Fig. 1), through screens into a suction well. From the well it is pumped through a Venturi meter into the receiving compartment at the Purification Works. Here, it is divided into approximately two equal parts. One part without treatment flows directly to the baffled mixing tanks and the other part, to which is added the entire quantity of lime, soda-ash, and alum solutions necessary to soften and clarify the entire supply, flows to mechanical mixing tanks.

The portion of water receiving the chemical charge is mixed for about 10 min. in square tanks provided with rotating paddles. During this time chemical reactions between the chemicals added to the water and the mineral salts already present in the water are effected. This reaction results in the formation of an extremely heavy gelatinous precipitate, which is largely due to the mass action effect of over-treatment. Under these circumstances the magnesium compounds present in the water are readily precipitated. Lime combines with magnesium to form magnesium hydroxide, a bulky gelatinous precipitate, which coagulates as readily as the precipitates formed by sulfates of alum or iron. It is, however, extremely difficult to precipitate magnesium unless excess lime is applied. This is the reason why separate treatment is used.

This over-treated portion of the water, with floc already formed, joins the other portion of untreated raw water at the entrance of the baffled mixing tanks, and the two are mixed together for about an hour. Floc formation having already been started, further floc-forming reactions take place readily in the baffled mixing tanks. The size of the floc grows as the water rolls over and under the baffles.

From the baffled mixing tanks the water flows to the settling basins (Fig. 2). They are six in number. The mud and precipitated chemicals settle to the bottom of these basins, practically all of it being removed in the first two. It is necessary to wash these basins every three or four weeks. The last four basins are washed only once or twice a year.

Just before the clear softened water leaves the settling basins, carbon-dioxide gas is applied in order to neutralize any excess lime. It also converts the normal carbonates to bi-carbonate form. This reaction stabilizes the water, and prevents after-reactions, or deposits, in the filters or distributing system.

After carbonation, the water discharges from the settling basins into the filters. Here, it is filtered through 24 in. of sand and 18 in. of gravel. As it discharges from the filters into the filtered water reservoir a small quantity of chlorine gas is added, the quantity averaging only 0.15 parts per million. The water meets the U. S. Treasury standards for bacterial purity without the chlorine, but the small quantity mentioned is added as a safeguard. From the filtered reservoir the water is pumped by means of high-lift pumps (Fig. 3) into the distributing system to the consumers.

Advantages of Lime Treatment

Lime is added to water primarily to soften it, but it has been discovered that, when added in sufficient quantity to produce residual or excess lime

(caustic alkalinity), it kills intestinal and pathogenic bacteria, rendering the water safe bacterially.

The lime-softening process also increases the efficiency of sedimentation because better coagulation is effected. The bulk of the precipitate formed by the softening reaction is about twenty times as great as could be obtained by ordinary coagulation processes. This large, bulky precipitate of calcium carbonate and magnesium hydroxide entangles fine particles of suspended matter, bacteria, and other impurities in the water, and settles very rapidly.

Lime-softened and settled water is low in turbidity, color, and organic matter, these having been absorbed or dragged down mechanically by the precipitates. In addition, the water will be free from objectionable gases, such as carbon dioxide and hydrogen sulfide, these having been neutralized. The riddance from bacteria, turbidity, iron, color, organic matter, and objectionable gases by lime treatment, lessens the cost of filtration because filter runs are long, and filtration areas may be reduced.

Lime softening is well adapted to the treatment of a polluted hard water, such as that at Columbus. It is possible during flood times, when tastes and odors are feared, to produce a satisfactory potable water without having to depend on chlorine to kill bacteria.

Application of Chemicals to Water

The lime, soda ash, and alum are now applied to the water at practically one point. When the plant was first put in operation the lime was applied to approximately 25% of the raw water, which was diverted through saturators in order to make a saturated lime water containing 60 grains of calcium oxide per gal. This saturated lime water was then mixed with 50% more of the raw water, making a total of 75% lime-treated water. The soda ash was mixed with the remaining 25% of the raw water for about 20 min. and then mixed with the lime-treated water. This was given up because it was found to be extremely difficult to produce a uniform saturated lime solution.

It was also discovered that considerable lime was lost in the sludge, which deposited in the bottom of the saturators. There was no noticeable advantage in treating the water separately with lime and soda ash and then mixing the two waters together. The resultant reaction is the same as when they are added together. The process was simplified by applying the chemicals at one point, but the split treatment is still used because the hardness of a given quantity of raw water with a given quantity of chemicals will be reduced more than when added in the usual way.

Water not "Doped" with Chemicals

There is a general belief that the use of chemicals for softening water is not desirable. There is usually prejudice against it. Wherever water-softening plants have been built, this prejudice has been almost entirely overcome. The citizens of municipalities operating a water-softening plant, especially in those cities where water softening has been practiced for some years, now understand that the water is not "doped". They have learned there is less mineral matter or chemicals in the softened water than in the raw water.

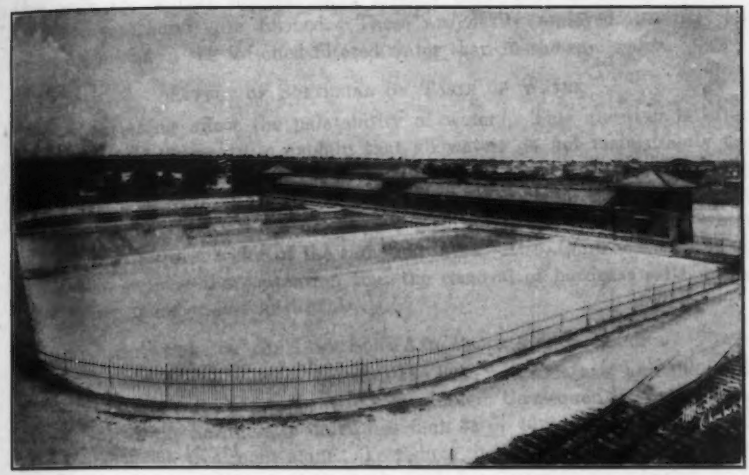

FIG. 2.—VIEW OF SETTLING BASINS, COLUMBUS, OHIO, PURIFICATION WORKS.

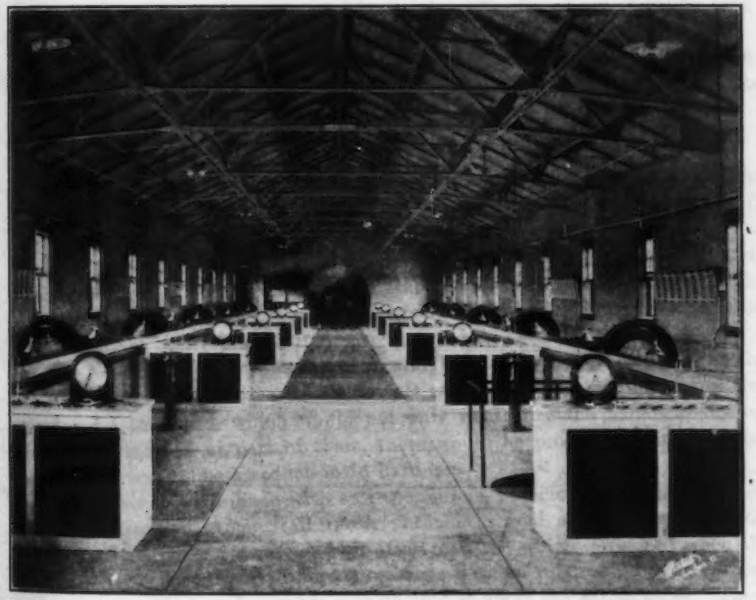

FIG. 3.—VIEW OF INTERIOR OF FILTER ROOM, SHOWING HIGH-LIFT PUMPS, COLUMBUS, OHIO, PURIFICATION PLANT.

The chemicals, which are added in order to soften and purify the water, combine with the soluble mineral salts already present in solution, and new insoluble compounds are formed. These are easily removed, leaving less mineral content in the softened filtered water than in the raw water.

Effect of Softening on Taste of Water

Does softening affect the palatability of water? This question is often asked and it is necessary to explain that all waters do not taste exactly the same. Water highly mineralized, or water that contains iron or sulfur compounds, has a more pronounced taste than water free from these compounds, but there should be no difference in the taste of softened carbonated water and that of a natural water of the same hardness. Also, there seems to be no evidence to support the contention that the removal of hardness salts makes water either more or less healthful.

Limitations of the Lime Soda-Ash Process of Softening

The product of the reaction between the lime and soda ash, and the hardness salts in the water, is not entirely insoluble. Consequently, it is impossible to reduce the hardness to much less than 35 or 40 parts per million with lime and soda-ash treatment alone. To reduce the hardness to this low figure an excess quantity of chemicals must be added.

It has been the experience at Columbus that when the hardness has been reduced to about 85 parts per million, the cost of further removal of hardness is proportionately higher. This is due to the difficulty of precipitating magnesium compounds.

Methods for further reducing hardness by high temperature, excess chemical treatment, split treatment, addition of aluminum salts and excess lime neutralized by carbonation have been described elsewhere by the writer.*

Lime Scale in Pipes and Filters

Lime-softened water is a saturated solution of calcium carbonate, and like any saturated solution is, of course, not stable. Difficulty has resulted from the deposition of calcium and magnesium scale in the filter sand at the plant and in the pipe system of the city. It has long been known that carbon dioxide applied to lime-softened water would stabilize it, because it re-acts with the slightly soluble non-carbonates and changes them to high soluble bi-carbonates, which are not easily precipitated.

For some time doubt was felt as to whether or not carbon dioxide was the best re-agent for corrective treatment. It was realized that if an inexpensive acid could be found, which would not only neutralize the mono-carbonates, but precipitate all or part of them, there would be produced a stable water and at the same time a reduction in hardness.

Experiments were made with sulfuric, silicic, boracic, oxalic, tartaric, and phosphoric acids. Among all these, phosphoric acid seemed best, but was given up because of its cost and an after reaction, which developed when the water was heated. Many different methods may be used to produce carbon-dioxide gas, and several methods may be used to apply the gas to water. These

* *The American City*, Vol. XXXVII, September, 1927.

were investigated over a period of seven years before starting the design of the carbonation plant.

Carbonation of a softened water makes it resemble natural water. If a sample of lime-softened water were submitted to a chemist for analysis, he could very easily tell that it was softened. The results of his analysis would show that the water was different from natural waters. If, however, a sample of lime-softened water that had been carbonated were submitted to him, he could not tell the source of the water from the results of his analysis. This water would be similar in all respects to any natural water of similar hardness.

It should be understood that the recarbonated water does not contain any free carbon dioxide, as only a sufficient amount to combine chemically with the normal carbonates is added, consequently carbonation of water as practised at Columbus should not make it corrosive.

The results of operation and quantities and cost of chemicals used, are given in Tables 2, 3, and 4. Table 5 shows the cost of treatment per 1 000 000 gal. and the cost of removing 1 part per million of hardness from 1 000 000 gal.

Fig. 4 shows the reduction in typhoid fever after filtration. There were five deaths from typhoid fever during 1926, three of which were resident and two non-resident. Assuming the population to be 285 000,* would give the city a total death rate from typhoid fever of 1.7 per 100 000, a resident death rate of 1.0 per 100 000, and a non-resident death rate of 0.7 per 100 000.

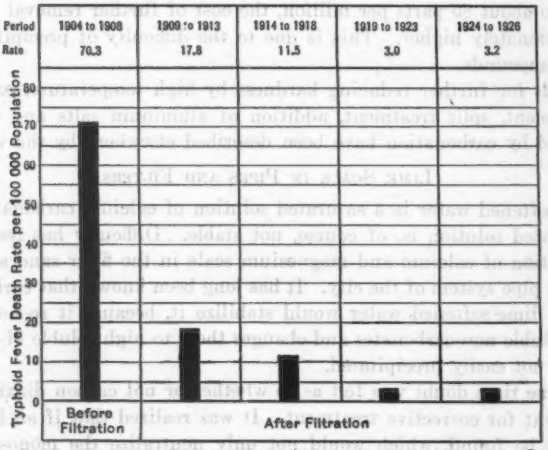

FIG. 4.—REDUCTION IN TYPHOID FEVER DEATH RATE IN COLUMBUS, OHIO, BEFORE AND AFTER FILTRATION.

IMPROVEMENTS AND ADDITIONAL FEATURES ADDED TO THE PLANT

Handling Lime.—Originally, the lime was handled and stored in canvas bags. In order to increase the storage capacity, and to lessen the labor and cost of handling the lime, a concrete storage bin was built. Lime is elevated into it by a conveyor. A cross-conveyor carries it to daily supply hoppers.

* Estimate made by Ohio State Bureau of Vital Statistics.

TABLE 2.—COMPARATIVE AND AVERAGE RESULTS OF OPERATION FROM 1909 TO 1926, BOTH INCLUSIVE.

Year.	Volume of water, in million gallons per 24 hours.	TOTAL HARDNESS, IN PARTS PER MILLION.			CARBONATE HARDNESS, IN PARTS PER MILLION.			NON-CARBONATE HARDNESS, IN PARTS PER MILLION.			MAGNESIUM, IN PARTS PER MILLION.		
		River water.	Filtered water.	Percentage removed.	River water.	Filtered water.	Percentage removed.	River water.	Filtered water.	Percentage removed.	River water.	Filtered water.	Percentage removed.
1909	14.3	253	93	63	150	40	73	102	53	48	21	10	53
1910	15.5	270	85	68	158	50	68	113	35	60	23	11	52
1911	15.6	245	84	61	150	45	71	95	38	60	21	10	51
1912	17.5	222	80	67	146	46	69	75	33	56	18	10	47
1913	18.3	271	88	67	160	61	62	111	28	75	23	12	46
1914	18.4	297	79	73	166	47	72	131	32	76	27	10	63
1915	17.7	279	88	69	170	51	70	108	36	67	23	9	59
1916	19.8	279	110	61	176	55	69	103	58	44	25	13	48
1917	21.6	278	125	55	152	45	70	126	81	36	22	12	46
1918	23.8	306	125	59	170	59	65	135	66	51	26	13	50
1919	22.7	278	106	62	160	51	68	113	55	51	22	11	50
1920	23.6	266	109	59	167	53	68	97	56	42	20	10	50
1921	21.4	260	100	63	162	54	67	107	46	57	24	11	54
1922	22.0	278	101	64	166	59	65	112	42	58	25	11	56
1923	23.9	265	95	64	156	54	66	109	41	62	22	10	55
1924	24.0	299	99	67	178	62	65	121	37	70	27	10	63
1925	26.4	298	92	69	161	59	63	137	32	77	27	9	67
1926	27.5	246	90	63	146	50	66	100	39	61	20	8	60
Average....	20.8	272	97	64	161	52	68	111	45	59	23	11	52

From the daily supply hoppers the lime drops into Richardson automatic weighing scales, which automatically weigh it into solution tanks from which it is fed into the water through a chemical feed regulator.

Alum Made at Plant.—It will be noted by examining Table 3 that whereas the cost of lime and soda ash has advanced since 1915, the price of alum is lower. The reason for this is the manufacture of alum at the plant.

Bauxite and sulfuric acid are boiled in lead-lined tanks until a basic solution of aluminum sulfate is obtained. The solution is then diluted with water (usually enough water is added to make 500 gal. of the solution, equivalent to 1 ton of 17% Al_2O_3 alum), and measured as needed into alum solution tanks. There it is diluted with sufficient water to make a standard solution, which is then applied to the water under treatment. By this process five distinct steps in alum making are eliminated—filtering, concentrating, crystallizing, grinding, and re-dissolving. The plant comprising lead-lined boiling tanks, an alum-measuring tank, one acid-measuring tank, storage tanks for sulfuric acid, a pulverizer for bauxite, conveying, elevating, and transmission machinery, bauxite storage bins and weighing device, piping, valves, and fittings, is shown in Fig. 5.

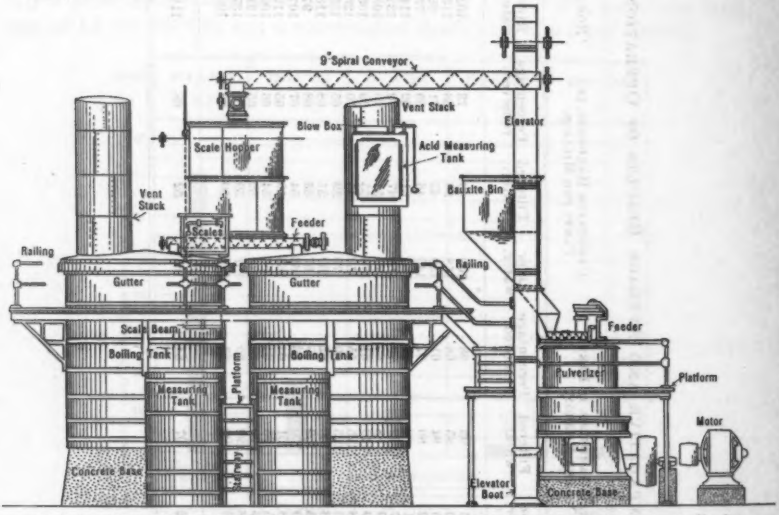

FIG. 5.—CROSS-SECTION OF ALUM PLANT, COLUMBUS, OHIO, WATER PURIFICATION PLANT.

Recarbonation Plant.—The gas-making and distributing plant consists of six principal parts (Fig. 6), a gas producer, a gas burner, a return tubular boiler, a steam-driven air compressor, a combination scrubber and drier, and a diffuser. In operation, coke is burned in the gas producer, which is a closed furnace with a controlled air supply furnished by a blower. The products of combustion pass from the gas producer to a gas burner, where they are mixed with air and burned to complete combustion under a boiler. The steam produced in this boiler is used to drive the air compressor and blower in order

TABLE 3.—Quantities and Cost of Chemicals.

Year.	Lime.		Soda-Ash.		Alum.		Bleach and Liquid Chlorine.	
	Tons.	Cost per ton.	Tons.	Cost per ton.	Tons.	Cost per ton.	Tons.	Cost per ton.
1909	2 467	$ 5.75	1 402	$17.50	624	$19.00*	..	.....
1910	3 081	5.80	2 164	17.50	423	18.00*	..	.....
1911	3 660	5.42	1 776	17.50	590	17.50*	..	.....
1912	3 269	5.27	1 583	15.20	895	17.15*	22	$ 27.50‡
1913	3 629	5.17	2 895	12.88	711	17.10*	17	27.80‡
1914	4 550	5.27	3 540	13.88	860	16.75*	14	29.20‡
1915	3 970	5.17	2 383	15.14	69	16.69*	22	34.80‡
1915	.....	.....	.....	.....	805	7.27†	..	.....
1916	4 550	5.17	1 975	62.00	823	20.00†	19	85.00‡
1917	4 206	7.05	1 835	60.00	943	10.00†	24	70.00‡
1918	4 810	9.22	3 005	50.60	1 113	14.62†	25	54.56‡
1919	5 149	10.21	2 748	40.50	1 156	14.58†	16	60.00‡
1920	5 307	11.90	2 149	43.60	1 304	15.52†	23	67.72‡
1921	5 166	12.05	2 679	42.54	985	13.39†	29	68.89‡
1922	5 487	10.98	2 891	28.81	1 275	12.50†	11	63.00‡
1922	.....	.....	.....	.....	.....	.....	4	112.00§
1923	5 594	10.62	3 246	30.95	1 173	11.20†	8	116.30§
1924	7 192	7.77	3 742	29.67	1 267	11.95†	6	101.85§
1925	7 769	7.25	4 303	29.86	1 384	11.66†	5	122.50§
1926	7 688	6.37	3 063	29.75	1 696	12.80†	6	157.50§

* Crystal alum purchased in the open market.
† Cost of materials. Sulfuric acid cost $14.63 per ton, f. o. b. Columbus, Ohio. Bauxite cost $14.96 per ton, f. o. b. Columbus, Ohio, 1926.
‡ Bleach.
§ Liquid chlorine.

TABLE 4.—Average Cost of Chemicals.

For period.	Lime.	Soda-Ash.	Alum.
1909 to 1915, inclusive.....	$5.41	$15.66	$16.18
1916 to 1921, inclusive.....	9.27	49.87	14.68
1922 to 1926, inclusive.....	8.60	29.81	12.02

TABLE 5.—Cost of Treatment per Million Gallons.*

Date.	Chemicals.	Labor.	Other.	Total.	Cost for chemicals per part removed, per million gallons.
1909	$ 9.77	$	$	$	$ 0.061
1910	11.40	4.99	1.39	17.78	0.062
1911	10.75	4.31	2.23	17.29	0.067
1912	8.95	3.64	0.84	13.43	0.063
1913	10.30	3.57	1.09	14.96	0.056
1914	12.53	3.79	1.00	16.32	0.057
1915	10.50	3.95	0.89	15.34	0.055
1916	22.70	3.44	0.83	26.97	0.134
1917	19.15	3.68	1.60	24.43	0.125
1918	27.60	4.60	3.35	35.55	0.153
1919	22.10	5.31	....	27.41	0.128
1920	20.80	5.86	1.35	28.01	0.132
1921	29.19	7.09	1.35	37.63	0.173
1922	22.13	6.69	1.46	30.28	0.125
1923	20.90	6.57	0.93	28.40	0.123
1924	21.17	6.79	0.93	28.89	0.106
1925	22.00	6.78	0.84	29.62	0.107
1926	16.38	6.42	1.11	23.91	0.105
Unweighted average.....	$18.15	$5.15	$1.25	$24.54	$0.102

* Of the cost of chemicals, 88% was for lime and soda-ash.

to make a closed cycle for the process. The plant is located in the boiler-room of the pumping station and the steam produced is connected to the main steam line. It has been found that a small amount of steam from the main steam line is required for operating the compressor and blower. The air compressor takes its suction from the boiler stack of the producer gas-burning boiler, through a scrubber and drier, and forces the washed gases through diffusers into the water to be carbonated. Making producer gas from coke, and burning it under a boiler, is perhaps the most economical method of producing carbon-dioxide gas. The steam generated is readily available, and it is possible to produce a gas containing as much as 17 or 18% of carbon dioxide, thus requiring an air compressor of only one-fourth the capacity needed if coke were burned in an open burner.

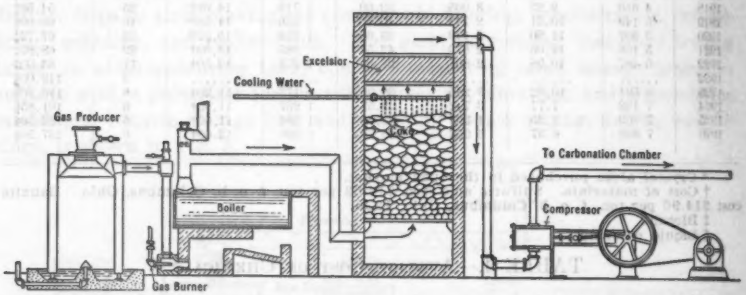

FIG. 6.—CROSS-SECTION THROUGH CARBONATION PLANT, COLUMBUS, OHIO, WATER-WORKS.

Plant Enlarged.—In 1923 the plant was enlarged by the addition of eight 3 000 000-gal. per day filters and the capacity of the filtered water reservoir was increased from 10 000 000 to 16 500 000 gal. With this enlargement it is possible to soften and filter 55 000 000 gal. per day at Columbus. This plant is the largest complete water-softening works in the world.

DISCUSSION

F. H. WARING,* M. AM. SOC. C. E.—An important part of the routine of operation at the Columbus Water Purification Plant should be stressed; namely, the practice of continual research on water-softening conducted both in the laboratory and on a plant-operation basis. Some items have been incorporated into large scale practice in the more recently constructed water-softening plants. Also, some have been used on a large scale at the Columbus plant. Mr. Hoover deserves great credit for the policy of making available to others the results of his experimentation and researches so that they may reap the benefit of his experiences.

The Use of Carbon Dioxide in Water-Softening Practice.—In 1922 it was demonstrated in the laboratory of the Columbus Water Purification Plant that the addition of carbon dioxide to lime-softened water was beneficial. A considerable portion of the calcium and magnesium carbonates was noticeably difficult to precipitate, as a result of which the sand grains of the filters and the water piping became coated. Some of the incrustation tended to pass beyond the filters and cause trouble in meters and hot-water systems. The introduction of carbon dioxide in the settled water passing to the filters destroys the semi-colloidal portion of the calcium carbonate precipitate by dissolving it as calcium bi-carbonate.

In municipal practice in the Middle West full utilization of this feature was not effected until the Defiance, Ohio, plant, designed by Nicholas S. Hill, Jr., M. Am. Soc. C. E., was placed in service in 1921.† Only a brief experience at the Defiance plant was required to demonstrate the efficacy of this feature of water-softening practice. From the start, the Defiance installation was a tremendous success, accomplishing the protection against incrustation of filters, piping, etc., already mentioned.

In Ohio, the custom has been established of an exchange of experiences among operators in charge of water treatment works by means of annual conferences sponsored by the State Department of Health. Thus, the news of the success at Defiance was rapidly spread within the State; and what was perhaps looked upon at first as experimental, namely, the carbon dioxide treatment, was soon viewed as an essential, accepted process in water-softening practice. Based on the early experiments at Columbus and the full scale practical demonstration on the part of the Defiance plant, Mr. Hoover incorporated the carbon dioxide treatment process in his designs for the municipal water-softening plant at Newark, Ohio. Following the Newark installation in 1924 there have been several carbon dioxide installations in Ohio, each based on previous experience, and, except for minor improvements, largely patterned after the Defiance layout. Delaware completed its plant in 1925; Piqua, Girard, and Greenville, in 1926; and Fostoria, Fremont, and Marion constructed new plants in 1927.

It is pertinent to emphasize that, in addition to preventing incrustation, the carbon dioxide treatment deprives the softened water of a taste given to

* Chf. Engr., State Dept. of Health, Columbus, Ohio.

† "The New Water Pumping and Purification Works at Defiance, Ohio," *Journal*. Am. Water Works Assoc., Vol. 7, No. 4, July, 1920.

it by the lime, described by some as a "limey" or caustic taste and by others as a peculiar flat taste. It is practically impossible to tell by taste the difference between any natural water of equivalent hardness and a softened water to which has been applied the proper amount of carbon dioxide gas. From a popular viewpoint, therefore, definite objection to the softening process as applied to municipal practice has been removed.

Bacterial Significance of Softening Treatment.—The promoters of the Columbus plant were pioneers in the field of water-softening. As stated by Mr. Hoover, very little opportunity was available for those in charge of testing station experiments to study fully the several phases of municipal water softening prior to the construction of the works.

A discovery of very great significance was made after the plant had been put into service. It was found that the water leaving the settling basins after softening treatment and prior to filtration was sterile whenever proper amounts of chemicals required for softening were applied. On further inquiry into the nature of this circumstance it was established that bacterial life was entirely destroyed and prevented if the application of the chemicals resulted in a final product that carried caustic alkalinity. Such alkalinity exists only when free hydrate, termed by some as an excess of lime, is present. It is of interest to note that a similar discovery was made almost simultaneously by Sir Alexander Houston, of London, England, who first showed the possible significance of the excess lime method of treatment from the bacterial standpoint.

As a result of the discovery the Columbus plant has been operating steadily from a water-softening standpoint and not with very much regard for the variations in bacterial content of the raw water being treated. From the bacterial standpoint it remained only necessary to be assured that sufficient lime was added so that the treated water was made slightly caustic.

This experience in the removal of bacteria at Columbus was capitalized by the City of Youngstown in 1918. Mr. Hoover was employed to adjust the treatment process at the Youngstown Water Purification Works so that caustic alkalinity would be obtained in the treated water. The Mahoning River at Youngstown, utilized as the source of raw water supply, carries a tremendous burden of bacteria. For example, the yearly average for 1926, measured in terms of *B. coli* per 100 cu. cm., was 9 100; and the maximum monthly average, likewise expressed, was 23 000. Following the introduction of the excess-lime treatment the final effluent of the plant has been acceptable from a bacterial standpoint without the assistance of chlorination or any other disinfection process. For example, the filtered water average for 1926, in terms of *B. coli* per 100 cu. cm., was 0.2; and the maximum monthly average was 0.6. Other water purification works in Ohio have utilized the excess-lime method for bacterial purposes solely, namely, East Liverpool, Ironton, Portsmouth, all on the Ohio River and carrying even heavier bacterial loads than those given for Youngstown. A prime reason for adopting the excess-lime method for bacterial effects at these cities was the necessity of eliminating and preventing chloro-phenol tastes which occurred occasionally, usually together with the high bacterial loads. The excess-lime treatment made it possible to avoid using chlorine

during the intervals that phenol was being experienced in the raw waters at these Ohio River plants.

If raw waters carrying heavy bacterial burdens must be utilized as a source of raw water supply, designing engineers may well give serious thought to the use of the excess-lime treatment to accomplish bacterial reduction, even if no water-softening benefits are desired.

Value of Recently Demonstrated Over-Treatment with Lime from a Softening Standpoint.—One of the features of the Columbus water-softening plant is the over-treatment of a part of the raw-water supply; that is, the raw water is divided into approximately equal parts, one part receiving all the softening re-agents; after thorough mixing this over-treated part is mingled with the part of the raw water that had received no treatment, and the whole is given efficient mixing before entering the settling basins.

The reason for over-treating a part of the supply is commonly known to the chemist by the term, "mass reaction". What really happens is that the extra heavy application of lime in the 50% part of the raw water causes the magnesium to come out of solution as magnesium hydrate, a fluffy flocculent precipitate. The magnesium will not come out of the water in such proportion by the action of the lime when only a slight caustic alkalinity is created by the latter; it takes an excess, or over-treatment of the lime to bring about the precipitation of the magnesium. This precipitation is practically complete for that part of the raw water thus treated in the Columbus plant.

Of course, with the processes originally provided it was early understood that such an excess of lime for all the raw water could not be tolerated if the final water was to be palatable and not have a harsh effect on the users of the supply. Therefore, the part that was over-treated, was neutralized, so to speak, with the remainder of the water receiving no treatment.

The introduction of the carbon dioxide process into water-softening practice has now made it practicable to conduct the mass reaction treatment, or over-treatment with lime, on a 100% scale as applied to the raw water. This is possible because the over-treatment in the lime application is easily compensated by carbon dioxide application and with the resultant precipitation of the excess hydrate alkalinity as crystalline calcium carbonate. In this procedure of water softening, the carbon dioxide is not added in an excess, which would tend to re-dissolve some of the calcium carbonate precipitate, but falls just short of such an amount.

As a result of this system of extra heavy lime treatment, followed by re-carbonation, it is possible to precipitate all the magnesium and to throw down the calcium carbonate in crystalline form. The water then coming to the filters carries practically no material to incrust the sand or piping.

Utilization of this feature on a plant scale of operation has been effected regularly since June, 1927, at the Piqua, Ohio, water-softening works. Reports from James Montgomery, Superintendent of the Piqua plant (formerly of the Columbus and Newark Water Departments), have demonstrated the practicability of this process for bringing about further softening of a water supply, coupled with a more pleasing final product than was heretofore possible. Following closely on the success of the Piqua plant, the treatment has

been installed regularly at the Greenville, Ohio, plant, under the efficient management of Mr. John Q. McGuire (also formerly of the Columbus Water Department). The total hardness of the Miami River water before treatment at the Piqua plant since June, 1927, has averaged 215 parts per million. The total hardness of the treated water since June has averaged about 50 parts per million. Mr. Montgomery estimates that the treated water would have averaged about 80 parts per million if the works had been operated on the usual plan that was in effect before June. The raw water supply at Greenville is taken from wells having an average total hardness of about 450 parts per million, of which approximately one-fourth represents incrustant hardness. The treated water since July, 1927, has averaged about 90 parts per million, whereas before July the treated water had a total hardness of about 120 parts per million.

It is of interest to note that the new softening procedure, using extra heavy lime treatment, made possible by carbon dioxide application, costs no more at the Greenville and Piqua plants than the old lime-soda method; but with the advantage, however, that the treated water is much softer.

The large scale experiences of Piqua and Greenville have demonstrated the additional value of the carbon dioxide process. As the result of the early experiences at Defiance and Newark, followed by the more recent experiences at Piqua and Greenville, a complete carbon dioxide treatment process has now (1927) been installed at Columbus, and probably before very long the new schedule of softening will be adopted there just as it has been at Piqua and Greenville.

The policy of freely exchanging with others the experiences of both research and plant operation has paid Columbus well; and at the same time it has extended great benefit to other municipalities. Mr. Hoover deserves commendation for the liberal application of this policy.

GEORGE W. FULLER,* M. AM. SOC. C. E.—Without intending to discuss this paper from a technical standpoint, it seems appropriate to say a few words in regard to the development of the art of water softening. In the autumn of 1903, when the late Rudolph Hering, M. Am. Soc. C. E., and the speaker were called to Columbus to assist in formulating a program for water supply and sewage disposal, it was thought that the greater difficulty with those two undertakings would relate to sewage treatment; hence, a sewage testing station was operated on a sizable scale for several months. As Mr. Hoover has stated, the tests that led to the basic design of the water purification plant were made in a somewhat haphazard manner so far as anything outside the laboratory itself was concerned. The work was done in carboys and small tanks. At that time there were only three sizable plants in existence, municipally operated; namely, at, McKeesport, Pa., Winnipeg, Man., Canada, and South Hampton, England, although there were quite a number of industrial plants for water softening.

The main purpose of the water-works improvements was to correct a very highly polluted water. Mr. Hoover speaks of 138 deaths per 100 000

* Cons. Engr. (Fuller & McClintock), New York, N. Y.

population from typhoid fever. It should be made plain that that situation has been changed so that there are now less than 5 deaths per 100 000 population per year, which is, of course, very gratifying from the public health standpoint.

In regard to the water-softening installation itself, it was not the ambition of the designers to produce a completely softened water, but rather to make the Columbus water supply approach in hardness that of the Great Lakes. In fact, the policy was expressed of operating the plant so as to produce a water of hardness equal to that of Lake Michigan.

In those days at Columbus, with a very hard water from galleries, the community was led to use many thousands of cisterns. The people were accustomed to rain water. They used their pressure-water supply to a substantial extent for operating water motors, whereby soft cistern water was pumped into the house for laundry purposes.

When the plant went in service it was found that a hardness of 120 parts per million, or about 7 grains per gal., did not suit the citizens. They wanted a softer water. Particularly, on Mondays, there was a constant ringing of the telephones at the City Hall, the Water Department Office, and the plant, to have a much softer water. That led to the experiences which Mr. Hoover has related, and which as the data in his paper will show, resulted in a constantly decreasing hardness down to a point of about 85 parts per million.

With that softening came some burdens, due to the separation of calcium and magnesium carbonates in excess of their natural solubility in water, which is, as the speaker recalls, about 35 parts per million in the absence of carbonic acid. These slowly forming crystalline products accumulated in the pipes, on the sand grains of the filters, in the household plumbing, and actually stopped the meters.

Mr. Hoover has energetically attacked these problems, taking a plant that was intended to be flexible, but not as well prepared in its design as it could be made now. He is entitled to great credit for the persistence with which he has kept up his research work. Every hour during this period of twenty years there has been an analysis made of the water at different stages through this plant, and he has thereby gained experience that entitles his opinion to a great deal of consideration. The speaker is glad to note that this pioneer work has been the forerunner of developments of the art which have been so significant and which have been found to be so well worthy of application in many places in Ohio and elsewhere.

A plant such as Mr. Hoover operates is in a certain sense a manufacturing plant, creating a product which the people like in their private homes, but which is also more suitable for manufacturing and for steam-raising purposes. The speaker is aware that a municipal plant does not always find it desirable or practicable to produce water that best serves various industrial purposes. There are two things here to be considered: First, how far is a municipality justified in producing a better water supply than that which serves, reasonably, the citizens and the water users, at the lowest cost; and, second, to what

extent may those who need water for special purposes expect support and consideration from the producers of the municipal supply?

What is referred to particularly, of course, is water for boiler feed. Without doubt, industrial plants in need of a very soft water can get it by local treatment of a portion of the public supply. When it comes to the question of what is the best manner of operating a municipal water supply, consideration must be given to those things which Mr. Hoover has described; that is, elimination of all the deposits that are annoying to those who control the plant, annoying in the meters and the service piping and household plumbing. On the other hand, the practice of recarbonation is complained of as increasing the corrosion problem when used as the supply to railroad locomotives and for steam raisers in general.

The speaker will not attempt to offer any fixed conclusions as between those two lines of thought. They do exist, however, and he is sure that Mr. Hoover will consider them in the future, as he has in the past, and that there will continue to come from Columbus a vast fund of helpful information.

ROBERT SPURR WESTON,* M. AM. SOC. C. E.—There are not many who realized with what insufficient data the designers of the Columbus plant had to work. Softening processes are about 100 years old, but for the most part they have been applied to small works in England that were supplied largely from wells. The experiences at plants like those at Canterbury and South Hampton, England, therefore, were not altogether applicable to the Columbus problem. Furthermore, the waters were largely calcareous rather than dolomitic and, consequently, difficulties due to slimy deposits of magnesia, and poor crystallization of the precipitate, were not so vital as those at Columbus in the early days.

The plants in the United States designed for industrial softening expressed nothing that was novel in the way of treatment and nothing that was based upon modern physical chemistry. The Columbus designers supplemented the data with the meager available results of experiments made in bottles on a very small scale. Yet, in spite of all difficulties, John H. Gregory, M. Am. Soc. C. E., and his advisers succeeded in designing a plant so flexible that it is operating to-day (1927), with some changes to be sure, but such changes have not vitiated the efficiency of the original design in any way.

It is fortunate that Mr. Hoover has been able to conduct his research work as well as he has. Another fortuitous circumstance is a rapidly changing river, which made necessary the employment of a man of Mr. Hoover's character and caliber to supervise the operation of the plant. One could see on visiting the plant in its early days, when the chalky incrustations on the sand grains were a serious detriment to its operation, that the management was hard put to get good results. Failure was often threatening; yet success was attained.

The work of Mr. Hoover in connection with the recarbonation of water has been mentioned, and one can only add praise of his genius to the com-

* Cons. Engr. (Weston & Sampson), Boston, Mass.

mendations of others. Mr. Hoover is also to be commended for the introduction of what almost every one engaged in water-works practice calls "Hoover alum", which effected, not only an economy of the first order from the standpoint of the Columbus plant, but also a much lower price for sulfate of alumina for other plants.

The foresight of those in charge of the Columbus plant in providing for excellent management and in keeping records which are accurate and informing, is to be greatly praised, and all who have had to consult these records appreciate the care and the skill with which they have been collected.

The tendency of modern standards of quality is toward a softer water. A hardness of 60 to 80 parts per million, the goal of most lime-soda plants, is failing to give satisfaction to critical consumers. In many cases, this means going beyond the lime and soda processes and using zeolite softeners, which can remove all the hardening constituents. The Ohio Valley Water Company has recently installed a large plant of this type at McKees Rocks, Pa., by which the hardness is being lowered to a degree that is more pleasing to the consumer than 60 or 80 parts per million, which, in Massachusetts by the way, is considered an extremely hard water. In Brookline, Mass., the water is called hard at 40 parts per million. The average hardness of the water in Boston, Mass., is 12 to 18 parts per million. The standards will surely rise and the combination of the lime-soda and zeolite processes for hard waters will come into more general use, especially where non-carbonate hardness is high and where salt, for the regeneration of the zeolite, is cheap. Even Mr. Hoover may soon begin to compare the cost of salt for reducing the sulfate content of the hardness of the water at Columbus by the zeolite process with that of soda by the lime-soda process. Surely all engineers are greatly indebted to Mr. Hoover for his paper, and the work that has preceded it.

C. W. Foulk,* Esq.—As a matter of interest the speaker presents the following data to show the advantage of softened water to the average householder. The Columbus plant removes from 180 to 190 parts per million of hardness from the Scioto River water. This, if left in the water, would destroy about $6 worth of soap per year in the average household. (The household price of soap is taken at 10 cents per lb.) On the basis of 50 000 households in Columbus, $300 000 worth of soap would be wasted each year. This waste is eliminated by the softened water.

Considering the positive gains arising from the use of soft water, attention is called to the following items in the construction and operation of a house, which are needed in a hard-water city, but not in one with a soft-water supply:

Hundred-barrel cistern	$110
Electric water pump	120
Installing pump	20
Extra plumbing	50
Total	$300

* Prof. of Analytical Chemistry, Ohio State Univ., Columbus, Ohio.

This means that it costs $300 less to build a house in Columbus than to build the same house under the same conditions in a hard-water city. If interest and depreciation are calculated, it is seen that there is an annual saving of from $25 to $30, because of the soft-water supply.

One more observation is worth making. Mr. Hoover has calculated that 1 ton of this Columbus water costs the housekeeper 4 cents. What other valuable commodity can be had at that price per ton, delivered?

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Paper No. 1717

UPWARD PRESSURES UNDER DAMS: EXPERIMENTS BY THE UNITED STATES BUREAU OF RECLAMATION*

By Julian Hinds,† M. Am. Soc. C. E.

With Discussion by Messrs. E. L. Chandler, B. F. Jakobsen, Charles Terzaghi, J. C. Stevens, F. W. Hanna, Ivan E. Houk, C. H. Howell, P. Wilhelm Werner, H. de B. Parsons, E. W. Lane, C. E. Grunsky, and Julian Hinds.

Synopsis

Uplift measurements have been made by the United States Bureau of Reclamation on three gravity diversion dams and one gravity storage dam. Two of the diversion dams, the Colorado River Dam, on the Grand Valley Project, in Colorado, and the Percha Dam, on the Rio Grande Project, in New Mexico, are founded on gravel. The other diversion dam, the Willwood Dam, on the Shoshone Project, in Wyoming, is founded on sandstone and shale, thoroughly grouted. The gravity storage dam, the recently completed American Falls Dam on the Minidoka Project of Idaho, is built on a columnar basalt foundation, also thoroughly grouted.

The measurements at the Colorado River and Percha Dams indicate that the materials deposited above a dam built on porous foundation act somewhat as a filter. When the filter surface is thoroughly seasoned, it offers considerable resistance to the passage of water, and the uplift pressure is partly relieved. When the filter surface is disturbed, as in time of flood, the uplift pressures are greater. The results are too indefinite to permit an attempt to prove, or disprove, any of the theories of flow under dams of this type.

Note.—The Special Committee on Irrigation Hydraulics selected the subject of "Water Movement and Pressure Under Dams" as one of ten for study and research. This paper was submitted to the Committee by the author and the Committee recommended its publication. (See Progress Report of the Committee, *Proceedings*, Am. Soc. C. E., March, 1927, Society Affairs, p. 121.)

* Published in March, 1928, *Proceedings*.

† Care, Dept. of Water and Power, Los Angeles, Calif.

The worst uplift conditions occur with a freshly disturbed filter surface, and experimental data taken under any other circumstances should be used with care. A limited area of escape at any point below the filter surface as at the Colorado River Dam, causes an increase in pressure above the constriction, and a decrease below.

At the Willwood Dam there is, generally, no evidence of appreciable uplift for that portion of the dam founded on shale. For sandstone the uplift pressures vary with the up-stream and down-stream water levels, in a reasonably regular way, and the uplift appears to vary, roughly as a straight line, from full tail-water depth at the toe to full tail-water plus one-fourth the difference in tail-water and head-water elevations at the heel.

The results at American Falls are generally similar to those at Willwood. Although there are no shale areas at this site there are a few apparently water-tight holes. The uplift pressure intensities are on the average slightly higher, relatively, than at Willwood.

Colorado River Dam

The type of design and general conditions at the Colorado River Dam are shown in Fig. 1, while the general appearance of the structure is shown in the photograph, Fig. 5. The design contemplated an open gravel foundation throughout, although it was known that rock could be reached at the ends of the structure. When the cut-off trench was opened, the gravel in a large portion of the river bed was found to be fairly tightly cemented, but not of sufficient hardness to justify changing to a standard type of gravity dam. The dam was finally founded on the upper strata of loose gravel, the up-stream cut-off being carried to tighter material except for a short distance in the vicinity of Pier *B*, where the open gravel extended to an undetermined depth, and across the sluice-way, where the percolation distance is such that a deep cut-off was not considered essential. The gravel in the river bed is mostly rather coarse with only a small percentage of sand.

The ogee crest of the dam is divided into six parts, each controlled by a 70 ft. by 10 ft. 3-in. roller crest. An additional 60 ft. by 15 ft. 3-in. roller controls a sluice-way at the west end of the dam. The total height from the top of the down-stream apron to the top of the rollers in the closed position is 18.25 ft., which is the maximum possible unbalanced head. The dam was completed in 1915.

Pier *B* is over the deep gravel portion of the foundation and is provided with two pipes for determining uplift pressures. These pipes are situated as shown in the pier detail, Fig. 1. Details of the pipe and its setting are shown in Fig. 2. One pipe was also installed in Pier *G*, its position being intermediate between the upper and lower pipe locations shown for Pier *B*.

A large number of readings have been recorded from these pipe wells, daily observations having been made throughout several irrigation seasons. Some of the results are given in Table 1. The data shown cover the full range of variations in head, and are typical.

Fig. 3 shows graphically the daily fluctuations in water levels from April 1 to October 30, 1922. A study of the relations shown on this diagram is interesting. The up-stream water level, E_1, is controlled by the roller gates, and is more or less independent of other factors. The tail-water level, E_4, depends on the flow passing over the dam. The fluctuations in the water level in the lower well, E_3, closely follow the changes in the depth of tail-water. The depth in the upper well, E_2, changes promptly with a change in the up-stream water level, and appears to respond more slowly to general changes in the height of tail-water.

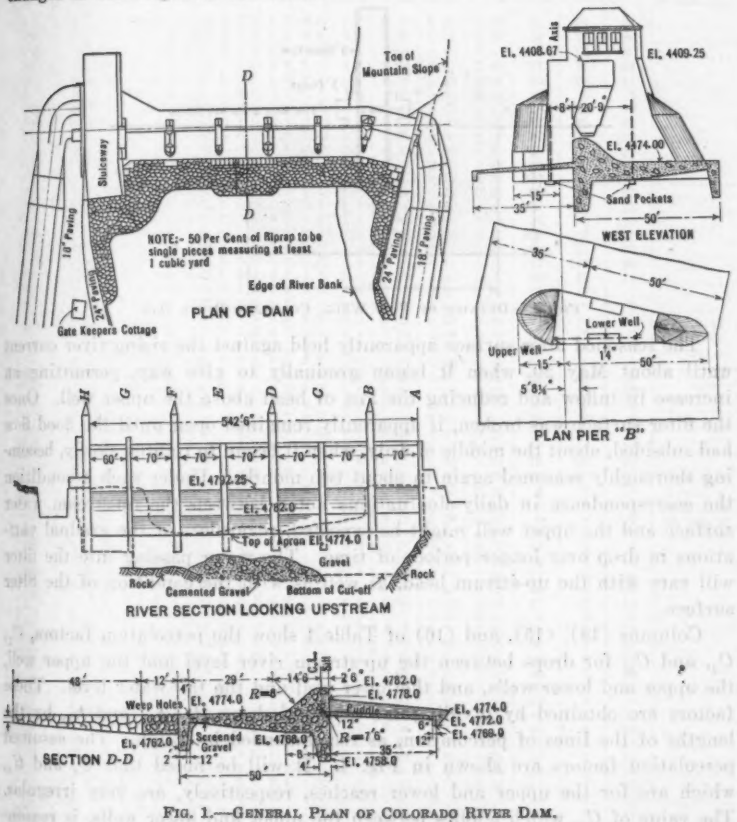


FIG. 1.—GENERAL PLAN OF COLORADO RIVER DAM.

It will be noted that on April 20 a gradual rise in tail-water depth began. The depth in the lower well immediately increased and followed the fluctuations in tail-water levels in detail. The upper well continued to respond in detail to changes in the up-stream level, but did not reflect the change in tail-water level until about thirty days later, when a general rise began. A similar delay occurred when the tail-water started to lower about June 15.

Although the general relation of the two upper curves in Fig. 3 vary widely, the daily fluctuations correspond in a remarkable way.

No entirely adequate explanation of the apparently delayed action of the upper well has been offered. It is probable that this effect is produced by changes in the mechanical or bacterial condition of the up-stream silt blanket, rather than by any hydraulic consideration. Prior to April 20 the discharge had been moderate and steady for a time and the silt bed above the dam, acting as a filter for the percolating water, had probably become well seasoned, causing an appreciable loss of head at the point of entrance.

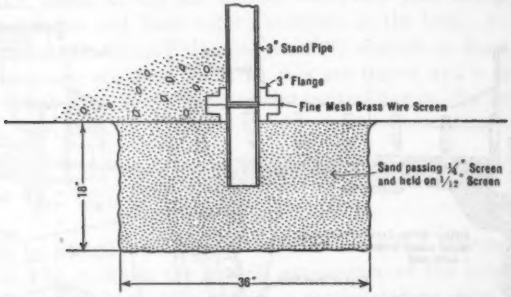


FIG. 2.—DETAILS OF PIPE WELL, COLORADO RIVER DAM.

The seasoned filter surface apparently held against the rising river current until about May 20, when it began gradually to give way, permitting an increase in inflow and reducing the loss of head above the upper well. Once the filter surface was broken, it apparently remained open until the flood flow had subsided, about the middle of July, when it began to tighten slowly, becoming thoroughly seasoned again in about two months. Under such a condition the correspondence in daily fluctuations noted between the up-stream water surface and the upper well might be expected, regardless of the gradual variations in drop over longer periods of time. The water passing into the filter will vary with the up-stream head, as well as with the condition of the filter surface.

Columns (14), (15), and (16) of Table 1 show the percolation factors, C_1, C_2, and C_3, for drops between the up-stream river level and the upper well, the upper and lower wells, and the lower well and the tail-water level. These factors are obtained by dividing the observed drops, h_1, h_2, and h_3, by the lengths of the lines of percolation, as recommended by Bligh. The assumed percolation factors are shown in Fig. 4. It will be noted that C_1 and C_3, which are for the upper and lower reaches, respectively, are very irregular. The value of C_2, which applies between the upper and lower wells, is reasonably consistent, except for measurements made before the rollers were put into service, and when the drop from head-water to tail-water was low. Because the total drop from head-water to tail-water varies, with the percolation distances constant, it is not logical that the percolation factors remain constant, but they might ordinarily be expected to remain proportional to each other, which they fail to do.

TABLE 1.—Measured Uplift Under Colorado River Dam.
(See Figs. 3 and 4.)

Year.	Month and day.	E_1.	E_2.	E_3.	E_4.	h_1.	h_2.	h_3.	H.	$\frac{h_1}{H}$.	$\frac{h_2}{H}$.	$\frac{h_3}{H}$.	C_1 $40+h_1$.	C_2 $43+h_2$.	C_3 $22+h_3$.	Remarks.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	
1916	2- 8	83.4	82.3	79.6	78.7	1.1	2.7	0.9	4.7	0.23	0.58	0.19	36	16	24	.....
1916	3-30	83.4	81.8	80.1	79.3	1.6	1.7	0.8	4.1	0.39	0.41	0.20	25	25	28	.....
1916	4-22	84.6	83.1	81.0	80.2	1.5	2.1	0.8	4.4	0.34	0.48	0.18	27	20	28	.....
1916	4-26	85.3	83.5	81.6	80.7	1.8	1.9	0.9	4.6	0.39	0.41	0.20	22	23	24	.....
1916	5- 3	85.9	83.9	82.1	81.4	2.0	1.8	0.7	4.5	0.44	0.40	0.16	20	24	31	.....
1916	5- 5	85.7	83.7	81.7	81.3	2.0	2.0	0.4	4.4	0.45	0.46	0.09	20	22	55	.....
1916	5- 8	87.0	85.0	83.2	82.9	2.0	1.8	0.3	4.1	0.49	0.44	0.07	20	24	73	.....
1916	5-10	88.0	86.5	84.4	83.7	1.5	2.1	0.7	4.3	0.35	0.49	0.16	27	20	31	.....
1916	5-25	84.7	84.1	82.6	82.0	0.6	1.5	0.6	2.7	0.22	0.56	0.22	67	29	37	.....
1916	5-27	85.0	84.5	83.1	82.5	0.5	1.4	0.6	2.5	0.20	0.56	0.24	80	31	37	.....
1916	5-30	85.2	84.8	83.5	83.0	0.4	1.3	0.5	2.2	0.18	0.59	0.23	100	33	44	.....
1916	6- 2	85.7	85.5	84.4	84.2	0.2	1.1	0.2	1.5	0.13	0.73	0.13	200	39	110	.....
1916	6- 4	85.8	85.6	84.6	84.2	0.2	1.0	0.4	1.6	0.12	0.63	0.25	200	43	55	.....
1916	6- 9	86.6	85.4	84.2	83.9	0.2	1.2	0.3	1.7	0.12	0.70	0.18	200	36	73	.....
1916	6-11	86.3	86.3	85.4	85.2	0.0	0.9	0.2	1.1	0.0	0.82	0.18	8	48	110	.....
1916	6-20	86.7	86.5	85.6	85.4	0.2	0.9	0.2	1.3	0.15	0.70	0.15	200	48	110	.....
1916	6-24	85.6	85.4	84.0	83.7	0.2	1.4	0.3	1.9	0.11	0.73	0.16	200	31	73	.....
1917	6-16	92.0	89.1	88.7	88.1	2.9	0.4	0.6	3.9	0.75	0.10	0.15	14	110	36	.....
1917	7-15	87.7	87.2	83.9	82.7	0.5	3.3	1.2	5.0	0.10	0.66	0.24	80	13	18	.....
1917	8-15	87.1	85.9	82.0	78.7	1.2	3.9	3.3	8.4	0.14	0.47	0.39	33	11	7	.....
1917	9-15	87.3	86.9	81.7	79.2	0.4	5.2	2.5	8.1	0.05	0.64	0.31	100	8	9	.....
1917	10-15	86.8	85.3	81.0	78.6	1.5	4.3	2.4	8.2	0.18	0.53	0.29	27	10	9	.....
1917	11-15	87.0	85.8	81.1	78.6	1.2	4.7	2.5	8.4	0.14	0.56	0.30	33	9	9	.....
1917	12-10	86.0	84.5	80.4	78.3	1.5	4.1	2.1	7.7	0.19	0.54	0.27	27	11	10	.....
1918	4- 1	87.5	86.9	81.6	78.9	0.6	5.3	2.7	8.6	0.07	0.62	0.31	67	8	8	.....
1918	6- 1	87.6	87.3	84.0	83.0	0.3	3.3	1.0	4.6	0.07	0.71	0.22	133	13	22	.....
1918	7- 1	87.5	87.1	83.6	81.7	0.4	3.5	1.9	5.8	0.07	0.60	0.33	100	12	12	.....
1918	8- 1	86.9	85.9	81.2	80.3	1.0	4.7	0.9	6.6	0.15	0.71	0.14	40	9	24	.....
1918	9- 2	87.6	86.8	81.1	78.2	0.8	5.7	2.9	9.4	0.09	0.60	0.31	50	8	8	.....
1918	10- 1	87.8	85.3	80.9	78.7	2.5	4.4	2.2	9.1	0.27	0.49	0.24	16	10	10	.....
1918	11- 1	87.1	84.8	80.8	78.8	2.3	4.0	2.0	8.3	0.28	0.48	0.24	17	11	11	.....
1918	12-28	85.9	83.3	79.9	78.1	2.6	3.4	1.8	7.8	0.33	0.44	0.23	15	13	12	.....
1920	4-20	86.8	83.6	79.4	76.2	3.2	4.2	3.2	10.6	0.30	0.40	0.30	13	10	7	10:00 A. M.
1920	4-20	89.4	84.3	78.3	76.2	5.1	6.0	2.1	13.2	0.39	0.45	0.16	8	7	10	11:00 A. M.
1920	4-20	91.8	85.4	78.6	76.2	6.4	6.8	2.4	15.6	0.41	0.44	0.15	6	6	9	11:45 A. M.
1920	4-20	92.8	85.8	80.6	76.2	7.0	5.2	4.4	16.6	0.42	0.32	0.26	6	8	5	12:15 P. M.
1920	4-20	89.3	86.0	81.3	76.2	3.3	4.7	5.1	13.1	0.25	0.36	0.39	12	11	4	1:00 P. M.
1922	4- 2	88.3	85.9	80.9	79.4	2.4	5.0	1.5	8.9	0.27	0.56	0.17	17	9	5	.....
1922	4-10	91.6	88.3	81.7	79.8	3.3	6.6	1.9	11.8	0.28	0.56	0.16	12	6	12	.....
1922	4-20	91.6	88.0	81.1	79.0	3.6	6.9	2.1	12.6	0.29	0 55	0.16	11	6	10	.....
1922	4-30	91.4	87.3	82.4	81.2	4.1	4.9	1.2	10.2	0.40	0.48	0.12	9	9	18	.....
1922	5- 9	91.4	89.0	84.1	82.9	2.4	4.9	1.2	8.5	0.28	0.58	0.14	17	9	13	.....
1922	5-17	91.9	87.8	83.0	81.9	4.1	4.8	1.1	10.0	0.41	0.48	0.11	9	9	20	.....
1922	5-29	91.7	90.6	85.9	85.6	1.1	4.7	0.3	6.1	0.18	0.77	0.05	36	9	73	.....
1922	6- 5	89.7	89.3	84.4	83.6	0.4	4.9	0.8	6.1	0.07	0.80	0.13	100	9	23	.....
1922	6-10	91.5	90.3	85.6	84.9	1.2	4.7	0.7	6.6	0.18	0.71	0.11	33	9	31	.....
1922	6-20	91.0	90.5	84.2	83.4	0.5	6.3	0.8	7.6	0.07	0.83	0.10	80	7	22	.....
1922	6-27	91.4	90.9	83.0	82.2	0.5	7.9	0.8	9.2	0.05	0.86	0.09	80	6	22	.....
1922	7-10	91.1	90.4	82.6	80.9	0.7	7.8	1.7	10.2	0.07	0.76	0.17	57	6	13	.....
1922	7-18	92.0	91.1	81.9	80.0	0.9	9.2	1.9	12.0	0.08	0.76	0.16	44	5	12	.....
1922	7-26	89.7	88.9	81.3	79.7	0.8	7.6	1.8	10.2	0.08	0.74	0.18	50	5	12	.....
1922	8- 5	93.2	89.6	81.5	79.8	2.6	8.1	1.7	12.4	0.21	0.65	0.14	15	5	13	.....
1922	8-16	89.7	87.2	80.8	79.2	2.5	6.4	1.6	10.5	0.24	0.61	0.15	16	7	14	.....
1922	8-18	91.7	88.6	81.4	79.7	3.1	7.2	1.7	12.0	0.26	0.60	0.14	13	6	13	.....
1922	8-28	90.5	86.9	80.8	79.5	3.6	6.1	1.3	11.0	0.33	0.55	0.12	11	7	17	.....
1922	9- 4	92.2	87.1	81.2	79.8	5.1	5.9	1.4	12.4	0.41	0.48	0.11	8	7	16	.....
1922	9- 7	90.7	85.9	80.5	79.2	4.8	5.4	1.3	11.5	0.42	0.47	0.11	8	8	17	.....
1922	9-13	87.0	84.9	80.5	79.1	2.1	4.4	1.4	7.9	0.26	0.56	0.18	19	10	16	.....
1922	9-22	91.8	86.6	80.5	78.9	5.2	6.1	1.6	12.9	0.41	0.47	0.12	8	7	14	.....
1922	10-10	92.3	86.8	80.5	79.0	5.5	6.3	1.5	13.3	0.41	0.47	0.12	7	7	15	.....
1922	10-18	91.4	86.3	80.5	78.9	5.1	5.8	1.6	12.5	0.41	0.46	0.13	8	7	14	.....
1922	10-27	91.9	86.5	80.3	78.8	5.4	6.2	1.5	13.1	0.41	0.48	0.11	7	7	15	.....

The variations in these factors can be accounted for in a general way by considering the physical conditions in connection with Fig. 3. Water can flow under the up-stream apron of the dam across the full width of the river channel, but can escape in appreciable quantity only through the narrow gap of open gravel under Pier *B*. Likewise, after passing the restricted open gravel area under the cut-off, the flow is free to spread laterally, thus reducing

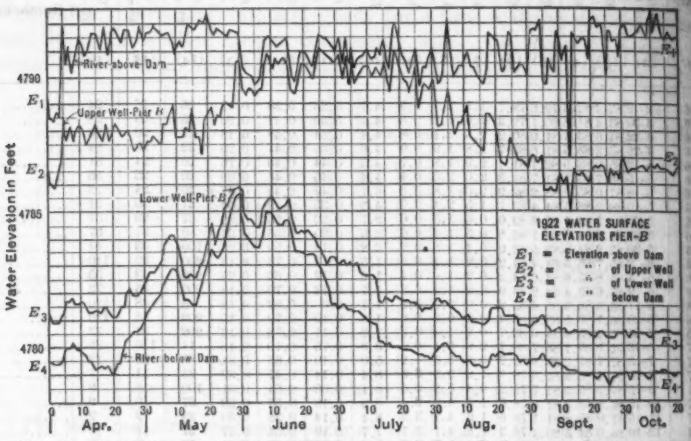


FIG. 3.—OBSERVATIONS OF WATER ELEVATIONS, AT PIER *B*, APRIL TO OCTOBER, 1922, COLORADO RIVER DAM.

the resistance to outflow. It seems logical, therefore, that the greatest loss should occur at the point of passing the cut-off wall. At times like June and July, 1922 (see Fig. 3), the great area open to easy infiltration causes the entrance loss to be small in comparison with the loss under the cut-off. In September and October the filter bed had tightened until, notwithstanding its

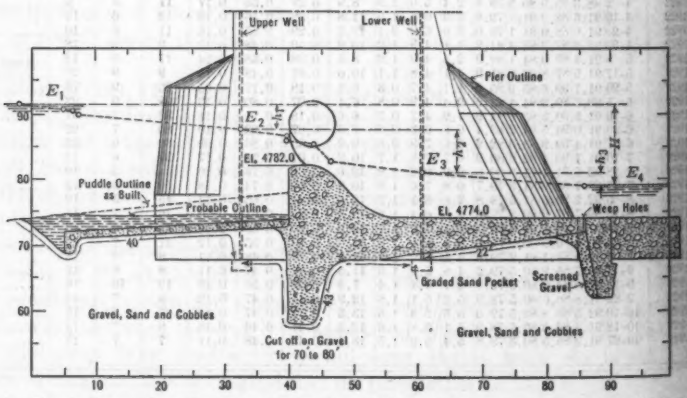


FIG. 4.—CROSS-SECTION OF COLORADO RIVER DAM, SHOWING HYDRAULIC GRADIENT COMPUTED FROM AVERAGE DATA FOR VALUES OF *H* EQUAL TO 12 FEET OR MORE.

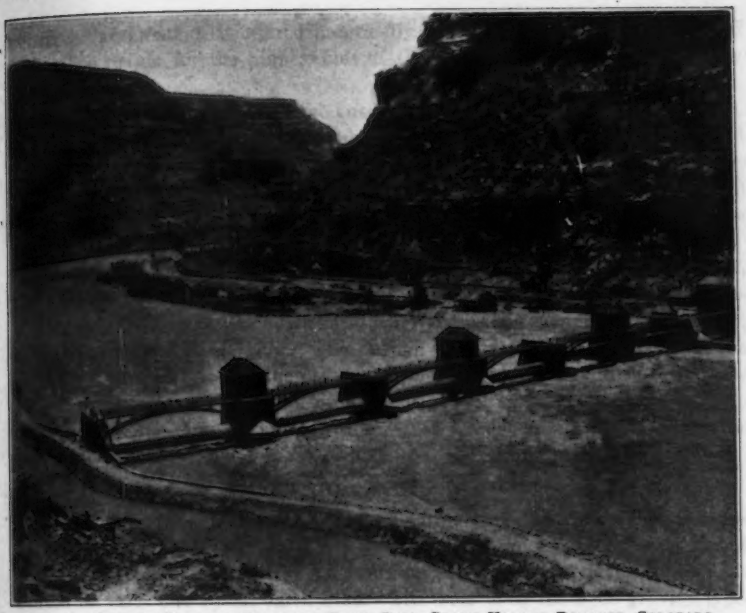
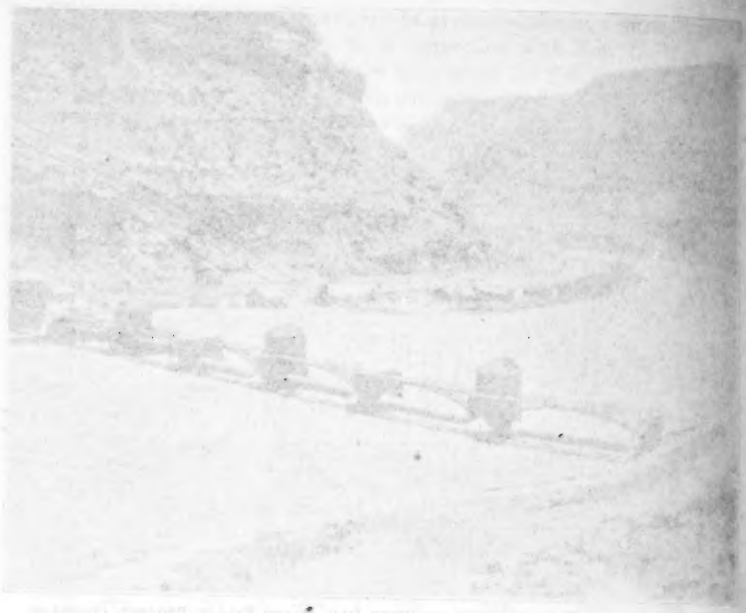


FIG. 5.—GENERAL VIEW OF COLORADO RIVER DAM, GRAND VALLEY PROJECT, COLORADO.

FIG. 6.—GENERAL VIEW OF PERCHA DIVERSION DAM, RIO GRANDE PROJECT, NEW MEXICO.



relatively large extent, it offered practically as much resistance as the cut-off wall. The reason for the high values of C_3 in May and June is not equally apparent.

If open gravel existed below the cut-off wall for the full length of the dam, the drop to the upper well would undoubtedly be materially increased, and that between the two wells would be decreased. Because of the irregular conditions existing at the Colorado River Dam, the data in Table 1 and Fig. 3 neither prove nor disprove Bligh's theory of a uniform pressure drop along the line of creep.

The one pressure well at Pier *G*, is just down stream from the cut-off wall. Observations indicate drops corresponding roughly with the drop to the down-stream well at Pier *B*. Conditions at Pier *G* are too complicated to permit an analysis of the data there. The cut-off wall is founded on rock on one side of the pier, and not on the other, and the long sluice structure introduces further uncertainty in the path of percolation.

A view of the site of the Colorado River Dam, looking down stream, is shown in Fig. 5.

Percha Dam

The Percha Dam is more strictly a floating type, no rock or impervious material being in the vicinity. Its general appearance is shown in Fig. 6 and its dimensions in Fig. 7. An overflow section, in the river channel, is flanked on each end by an earth dike. The weir is of the ogee gravity type, with free overflow crest. Heavy paving and rip-rap below the dam prevent scour and piping. Upward pressure is relieved a short distance below the toe of the gravity section by weep-holes through the rubble concrete apron. Percolation is retarded, and uplift reduced by an up-stream concrete apron, two concrete cut-off walls, and a line of steel sheet-piling under the up-stream cut-off. Construction was completed in 1917. No back-filling above the up-stream apron, to reduce leakage, was done when the dam was constructed, but the river bed promptly silted up, practically to the top of the weir.

TABLE 2.—Measured Uplift Under Percha Dam (See Fig. 9).

Year.	Month and day.	E_0.	E_1.	E_2.	E_3.	E_4.	E_5.*	E_6.	h_1.	h_2.	h_3.	h_4.	h_5.*	h_6.*	Total drop.	Remarks.
1918	3-10	03.80	03.25	03.25	99.25	98.75	97.65	96.45	0.55	—0.00	4.00	0.50	1.10	1.20	7.35	.....
1920	4-25	03.72	97.85	98.00	96.20	96.05	96.55	.....	5.87	0.15	1.80	0.15	—0.50	....	....	.....
1920	3-..	03.93	98.36	98.36	96.59	96.37	96.56	.....	5.57	0.00	1.77	0.22	—0.19	....	....	.....
1920	.....	04.25	99.46	99.56	97.56	97.36	97.06	.....	4.79	—0.10	2.00	0.20	—1.20	....	....	.....
1921	10-25	04.10	99.41	99.51	96.41	96.36	98.11	94.74	4.69	—0.10	3.10	0.05	—1.75	3.37	9.36	.....
1921	11-19	04.01	99.26	99.36	96.26	96.05	96.77	94.42	4.75	—0.10	3.10	0.21	—0.72	2.35	9.59	.....
1921	11-20	03.72	99.16	99.16	96.16	95.75	96.67	93.86	4.56	0.00	3.00	0.41	—0.92	2.81	9.86	.....
1921	11-21	03.71	99.01	99.01	96.71	95.60	96.57	93.84	4.70	0.00	2.30	1.11	—0.97	2.73	9.87	.....
1921	11-22	04.34	99.56	99.56	96.61	96.50	97.17	95.10	4.78	0.00	2.95	0.11	—0.67	2.07	9.24	Plotted.
1921	11-23	04.18	99.36	99.46	96.46	96.35	97.17	94.72	4.77	—0.10	3.00	0.11	—0.82	2.45	9.41	.....
1923	10-30	03.39	03.01	99.91	96.06	95.01	95.91	94.17	0.38	3.10	3.85	1.05	—0.90	1.74	9.22	.....
1923	10-31	03.39	03.21	00.06	96.16	95.01	95.91	94.17	0.18	3.15	3.90	1.15	—0.90	1.74	9.22	.....

* Possible leak in Well No. 5.

The material in the river bed grades from river sand with a small percentage of gravel at the surface to heavy boulders with some gravel and sand

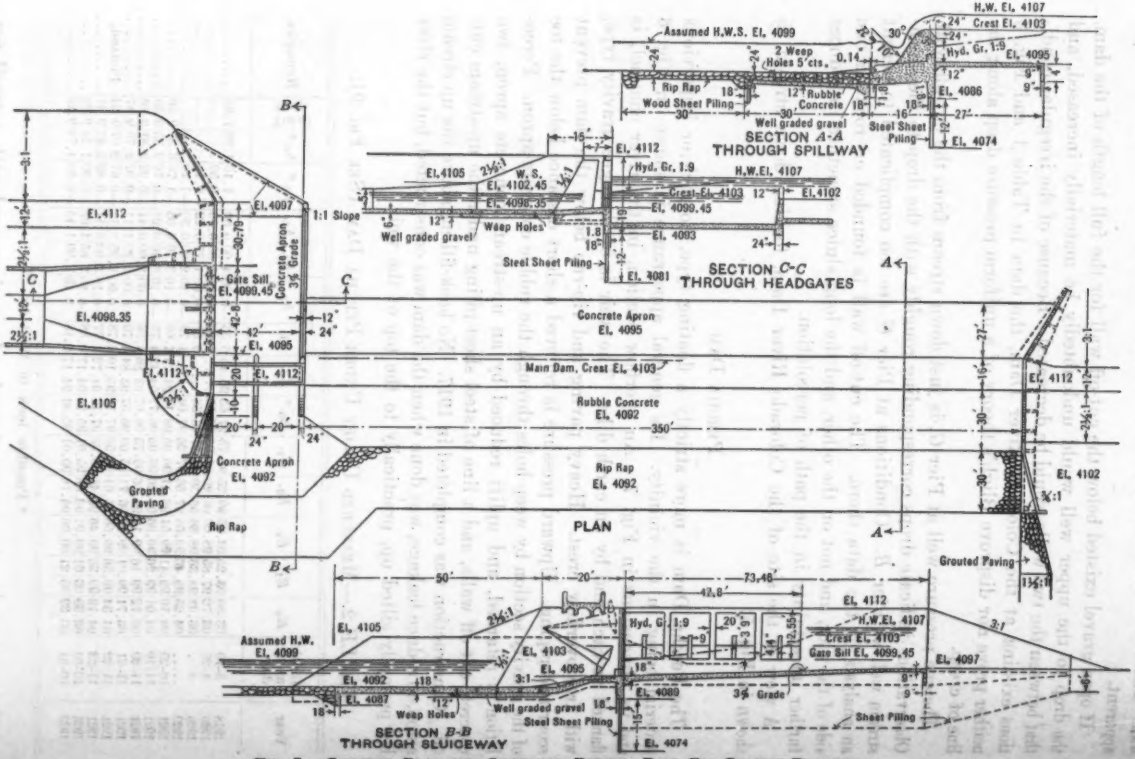


FIG. 7.—GENERAL PLAN AND SECTIONS, PERCHA DAM, RIO GRANDE PROJECT.

at the bottom of the steel sheet-piles. The material is graded and silted until it is reasonably resistant to percolation.

At the time of construction a system of pipes was installed for the measurement of uplift, as illustrated in Figs. 8 and 9. Uplift pressures have since been observed, as recorded in Table 2. The various notations, E_1, E_2, etc., and h_1, h_2, etc., have the same meanings (see Fig. 9) as in the case of Table 1. Inspection of Table 2 shows that except the first and the last two sets of data approximately one-half the total drop occurs between the up-stream water level and Well No. 1, which is just inside the up-stream apron cut-off. A large part of the remainder appears in h_3, which is the drop produced by the sheet-piling cut-off. Losses along the flat surfaces, between Wells Nos. 1 and 2, and Nos. 2 and 4, are small.

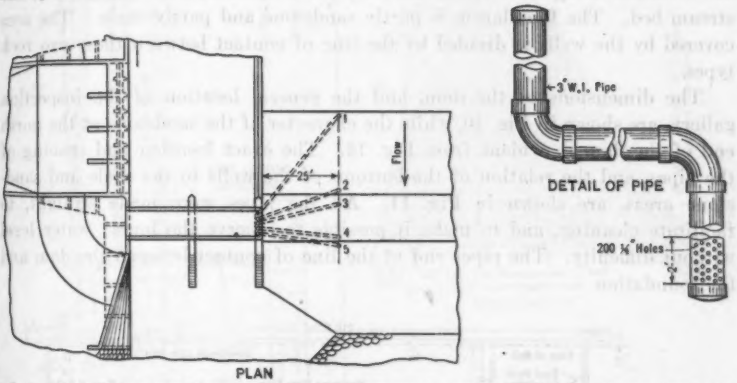


FIG. 8.—PIPE WELLS FOR MEASURING UPWARD PRESSURE, PERCHA DAM, RIO GRANDE PROJECT.

The first set of readings, and the last two, Table 2, show practically no loss at entrance. The first readings were taken soon after the dam was placed, and perhaps before the channel had completely silted and seasoned sufficiently to resist the entrance of water under the apron materially. The second set of readings was taken after the summer flood period, and it is probable that

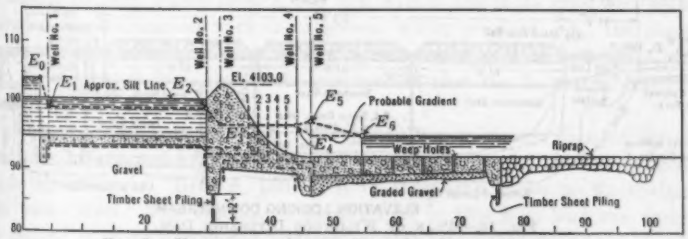


FIG. 9.—UNDERFLOW, PERCHA DAM, RIO GRANDE PROJECT.

the up-stream silt bed had been disturbed or partly removed. The entrance drop, h_1, was accordingly small at these times, the other drops being thereby materially increased. The first reading shows the loss to occur almost entirely at the two cut-off walls, the drop along the horizontal surfaces being small.

FIG. 7.—GENERAL PLAN AND SECTIONS, PERCHA DAM, RIO GRANDE PROJECT.
SECTION B-B THROUGH SLUICEWAY
El. 4074

The last two sets of data correspond reasonably well to Bligh's theory of a uniform drop along the line of creep, considering the steel sheet-piling to be water-tight.

The data presented are too limited to form the basis of any definite conclusion, but support, in a general way, the tentative conclusions drawn from the data for the Colorado River Dam.

Willwood Dam

The Willwood Dam (Figs. 10 and 12) is a low ogee gravity dam founded on rock. At the time of construction twelve pipes were placed in the concrete (Fig. 11) for determining the upward pressure under the dam. These pipes are arranged along three lines, relatively close together, near the center of the stream bed. The foundation is partly sandstone and partly shale. The area covered by the wells is divided by the line of contact between these two rock types.

The dimensions of the dam, and the general location of the inspection gallery, are shown in Fig. 10, while the character of the sandstone at the north end of the dam is evident from Fig. 13. The exact location and spacing of the pipes, and the relation of the bottoms of the wells to the shale and sandstone areas, are shown in Fig. 11. All the pipes were made straight, to facilitate cleaning, and to make it possible to observe the lowest water level without difficulty. The pipes end at the line of contact between the dam and the foundation.

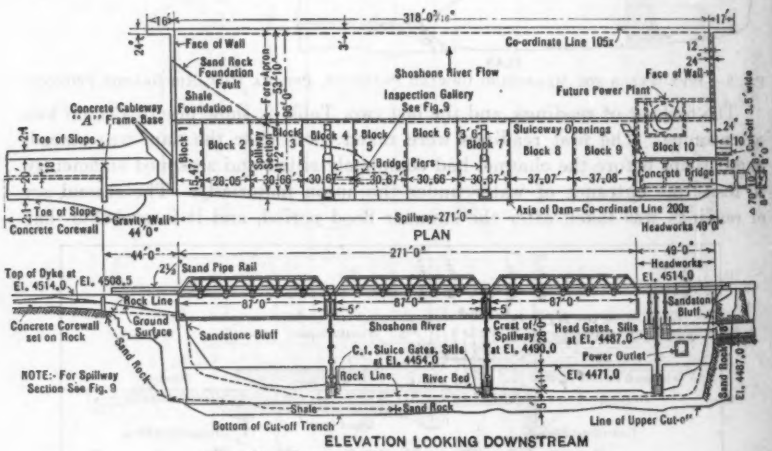


Fig. 10.—Plan of Willwood Diversion Dam.

The sandstone is somewhat broken and porous. The shale is reasonably good, and when protected from the air is amply strong for the expected bearing pressures. The foundation was thoroughly grouted along the up-stream cut-off. The results obtained to 1926 are shown in Table 3 and on Fig. 14. From

a study of Fig. 14, it appears that in the sandstone areas the uplift pressures vary in a regular way with the heights of the head- and tail-waters. Pressures in the shale areas are erratic.

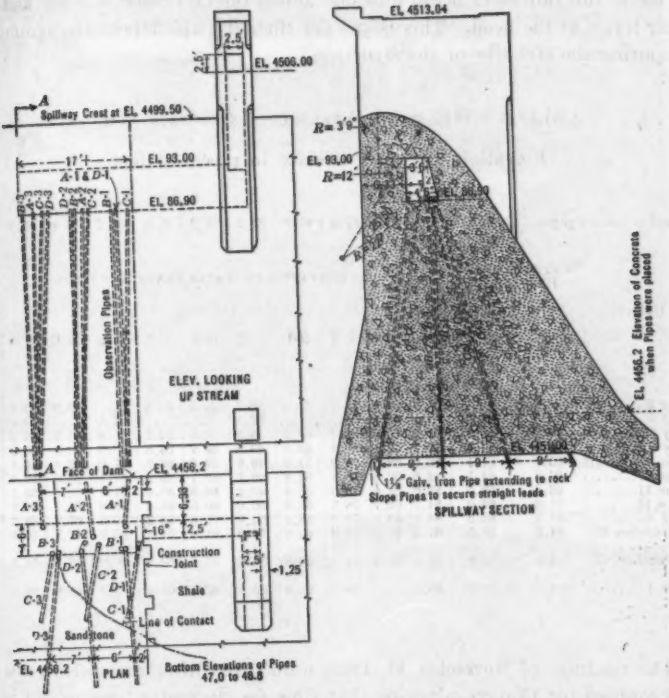


FIG. 11.—PLAN SHOWING LOCATION OF PRESSURE PIPES, WILLWOOD DIVERSION DAM.

Holes *B*, Line 1 and Line 2, are well within the shale area, and seem to have no communication with the reservoir water pressure. All the observed levels in these holes are below both the head-water and the tail-water, which can be accounted for only on the assumption that the holes are entirely or almost entirely water-tight.

Hole *A*, Line 1, is in shale, but is near the up-stream face of the dam, and may be reached by a crevice, which accounts for the evident connection with the reservoir. Hole *A*, Line 2, is in shale, and shows no fluctuation in the water level. The absence of fluctuations in this hole lead the observers to believe it to be obstructed. However, this was not definitely determined, and tightness may be due entirely to the shale bottom.

Hole *C*, Line 1, is in shale, but near the line of contact between the shale and sandstone. It appears to communicate with the pressures in the sandstone. Hole *C*, Line 3, is reported to be plugged, and no measurements for it are recorded.

Line 3 is entirely in sandstone, and shows a reasonably definite variation in uplift pressure as the head- and tail-water levels vary. The uplift pressure appears to vary, approximately as a straight line, from full tail-water depth at the toe to full tail-water depth plus one-fourth the difference in head- and tail-water levels at the head. This is smaller than the uplift pressure assumed in computing the stability of the structure.

TABLE 3.—WILLWOOD DIVERSION—INSPECTION GALLERY.

(Elevations of water surface in pipes, in feet.)

Elevation, top of pipes.....			87.2	87.1	87.2	87.0	87.2	87.2	87.1	87.4	87.4	87.5	87.4
Date.	WATER-SURFACE ELEVATION.		ELEVATION OF WATER SURFACE IN PIPES.										
	Reservoir.	Tail-water.	*A*-1.	*A*-2.	*A*-3.	*B*-1.	*B*-2.	*B*-3.	*C*-1.	*C*-2.	*D*-1.	*D*-2.	*D*-3.
1923:													
May 2.....	69.3	60.3	63.7	63.1	64.1	60.9	58.7	62.5	59.8	.....	63.0	63.4	62.0
May 9.....	73.0	60.4	.....	62.1	63.5	.....	52.1	.....	.....	61.6	.....	59.8	61.0
May 17.....	76.2	60.5	62.2	63.5	66.7	58.3	49.2	67.0	62.7	63.8	61.2	62.9	62.7
May 31.....	91.7	61.1	80.7	62.8	69.2	60.0	49.8	69.7	54.5	68.0	63.3	65.0	65.6
June 1.....	95.7	61.1	82.0	62.5	72.9	54.0	49.5	69.0	62.5	67.2	62.8	66.8	66.1
June 7.....	96.4	61.1	82.6	62.6	73.8	54.8	49.7	69.1	65.8	67.5	63.0	69.9	66.3
June 11.....	93.6	62.0	82.0	62.6	79.0	55.2	49.4	69.3	65.6	67.2	62.0	66.7	68.8
June 16.....	01.2	62.0	84.2	62.7	76.7	55.9	50.0	69.5	66.4	68.0	64.1	67.9	66.9
July 6.....	01.4	62.5	84.4	63.1	79.0	58.8	50.0	70.6	68.3	69.6	65.2	68.6	68.5
September 29.	84.3	59.5	67.6	62.7	73.3	60.6	51.0	61.8	61.1	65.7	62.3	66.0	60.8
1925:													
November 11.	64.8	58.8	66.5	68.9	74.4	63.8	50.0	65.6	63.8	59.0	56.7	56.7	61.1
1926:													
July 1.....	02.7	62.0	80.6	.....	66.3	63.6	50.5	65.6	63.2	63.2	63.6	61.6	61.4

The readings of November 11, 1925, were taken after the sluice-gates had been lowered for 15 days, allowing that time for the water level to fall in the wells. It should be noted that the water level in Holes *A* is high in all three lines. After these observations were taken, the water was artificially removed from all wells. Readings were again taken on July 1, 1926, after the gates had been closed since February 12. The readings were taken at high water, with the dam overtopped. The head-water had not, at any time since the unwatering of the wells, been more than 2 ft. higher than at the time of the observation.

The maximum uplift pressures observed at the Willwood Dam up to the present observations are plotted in Fig. 15. Uplift curves at Lines 1, 2, and 3 have been arbitrarily extended as straight lines from the values observed at the *A*-pipes to the maximum observed reservoir head at the up-stream edge of the dam; also, as straight lines from the values observed at the *D*-pipes to the maximum observed tail-water head at the down-stream edge. The latter is probably an accurate assumption. The former may indicate uplift pressures greater than those actually existing due to the thorough grouting of the foundation.

FIG. 12.—GENERAL VIEW OF WILLWOOD DAM, SHOSHONE PROJECT, WYOMING.

FIG. 13.—VIEW SHOWING CHARACTER OF SANDSTONE AT NORTH ABUTMENT, WILLWOOD DAM, SHOSHONE PROJECT, WYOMING.

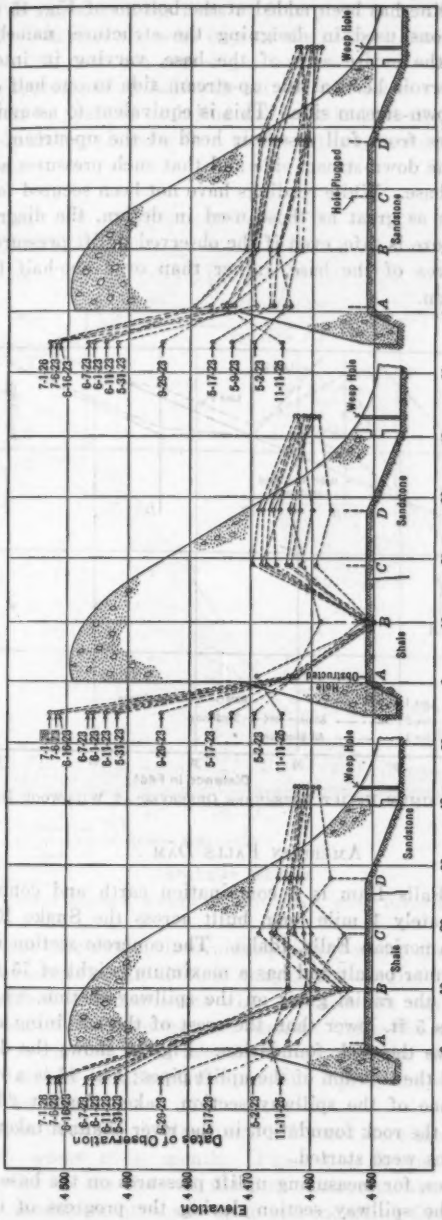


FIG. 14.—UPLIFT PRESSURES OBSERVED AT WILLWOOD DIVERSION DAM.

A full diagonal line has been added at the bottom of Fig. 15 to illustrate the uplift assumptions used in designing the structure, namely, that the pressures act over the entire area of the base, varying in intensity from one-half of full reservoir head at the up-stream side to one-half of full tail-water head at the down-stream side. This is equivalent to assuming that the uplift pressure varies from full reservoir head at the up-stream side to full tail-water head at the down-stream side and that such pressures act over one-half the area of the base. While readings have not been secured for head- and tail-water elevations as great as those used in design, the diagrams clearly show that the structure is safe, even if the observed uplift pressure intensities act over the full area of the base, rather than over one-half the area, as assumed in the design.

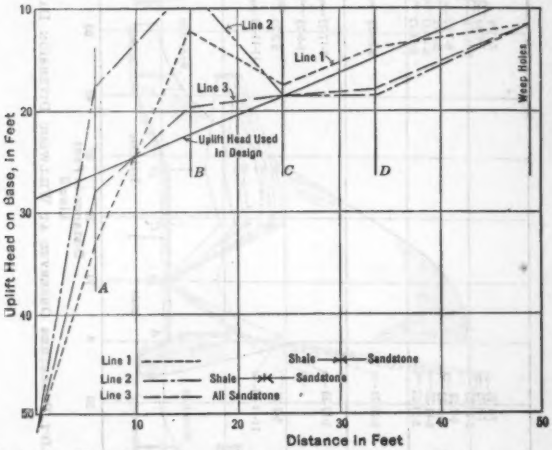


FIG. 15.—MAXIMUM UPLIFT PRESSURES OBSERVED AT WILLWOOD DAM.

AMERICAN FALLS DAM

The American Falls Dam is a combination earth and concrete gravity structure, approximately 1 mile long, built across the Snake River Valley, near the Town of American Falls, Idaho. The concrete section is built on a foundation of columnar basalt and has a maximum height of 75 ft. above the rock. The crest of the radial gates on the spillway section, which occupies the river channel, is 5 ft. lower than the crest of the adjoining concrete sections, or 70 ft. above the rock foundation. Fig. 16 shows the design of the spillway section and the location of the uplift pipes; Fig. 17 is a view showing the down-stream face of the spillway section, taken August 26, 1927; and Fig. 18 is a view of the rock foundation in the river channel taken just before concreting operations were started.

Twelve 3-in. pipes, for measuring uplift pressures on the base of the dam, were installed in the spillway section during the progress of construction.

These were arranged in three lines of four pipes each. One line was located at Station 9 + 00 near the west edge of the river channel; one was placed at Station 13 + 00, near the east edge; and the third was located at Station 11 + 10, or approximately half-way between the other two. Individual pipes were spaced 12 ft. apart in each line and were located as shown in Fig. 16, the up-stream pipe being placed 6 ft. 6 in. from the up-stream face of the dam in each instance.

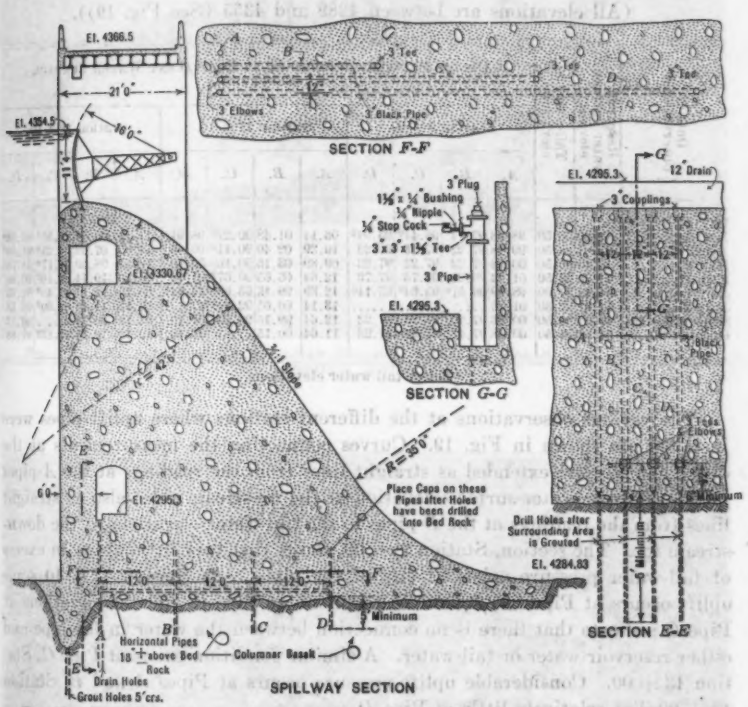


FIG. 16.—SPILLWAY SECTION, AMERICAN FALLS DAM, WITH PIPES FOR UPLIFT PRESSURE.

All the pipes were run to the drainage gallery where observations were made by reading pressures on an altitude gauge, or by sounding in the pipes. The pipes were placed in the concrete when the first lift was poured, leaving the lower ends about 6 in. above the rock surface. After the up-stream part of the foundation was grouted, holes were drilled through the vertical pipes approximately 5 ft. into the rock formation and the tops of the vertical pipes capped, as indicated on Fig. 16.

Observations were begun March 25, 1927, and were then taken at intervals of from two weeks to a month. Pipes not flowing were pumped out three days prior to the date of the first measurements. Table 4 gives the results of all measurements made up to and including October 20, 1927, uplift

pressures at the base of the dam being expressed as equivalent water-surface elevations. In Table 4 elevations below the tail-water level show that the water surfaces in the pipes were not being affected by either reservoir or tail-water pressures; or, in other words, that the foundation at these locations was tight on the dates such elevations were measured.

TABLE 4.—MEASURED UPLIFT ON BASE OF AMERICAN FALLS DAM.

(All elevations are between 4289 and 4355 (See Fig. 19)).

Date of observations.	Reservoir water surface elevation.	Tail-water elevation.	Uplift Pressure Expressed as Equivalent Water Surface Elevation.											
			Station 9 + 00.				Station 11 + 10.				Station 13 + 00.			
			A.	*B.*	*C.*	*D.*	*A.*	*B.*	*C.*	*D.*	*A.*	*B.*	*C.*	*D.*
1927:														
March 25.....	34.20	96.50	98.48	97.14	96.44*	95.94*	06.14	01.48	90.21*	98.91	14.15	06.64	94.55*	89.39*
April 15.....	37.88	96.50	99.48	97.22	97.23	97.23	10.39	02.40	90.11*	99.66	14.65	07.39	94.35*	91.69*
May 2.....	40.40	96.50	00.23	97.22	97.23	97.23	09.89	03.15	90.16*	00.16	16.15	98.39	94.17*	91.79*
May 17.....	44.76	96.50	01.23	98.22	97.73	97.73	12.64	05.63	90.67*	01.16	20.15	10.14	94.10*	96.36
July 1.....	54.46	97.00	98.99	96.51*	95.64*	95.14*	12.39	99.15	93.00*	93.64*	25.15	12.14	95.45*	01.13
July 21.....	54.31	97.30	01.23	.....	.....	.....	13.14	00.65	92.94*	.....	25.15	11.14	94.30*	02.13
September 19.	49.06	96.50	00.23	97.22	97.23	97.23	12.64	99.15	98.65	97.16	22.15	10.14	.....	03.13
October 20....	51.38	96.50	00.48	97.47	97.23	97.23	11.64	00.15	.....	97.16	22.15	10.14	93.63*	97.66

* Below tail-water elevations.

The various observations at the different stations where uplift pipes were installed are shown in Fig. 19. Curves connecting the measurements at the different pipes are extended as straight lines from the readings at the *A*-pipes to the reservoir water-surface elevation at the up-stream side; also as straight lines from the readings at the *D*-pipes to the tail-water elevations at the down-stream toe. The section, Station 9 + 00, shows that very little uplift in excess of tail-water pressure exists at any of the pipes. At Station 11 + 10 some uplift occurs at Pipes *A*, *B*, and *D*, but none at Pipe *C*. The observations at Pipe *C* indicate that there is no connection between the water in the pipe and either reservoir water or tail-water. A similar condition exists at Pipe *C*, Station 13 + 00. Considerable uplift pressure occurs at Pipes *A* and *B*, Station 13 + 00, but relatively little at Pipe *D*.

In Fig. 20 is shown a summary of maximum uplift observations at the American Falls Dam. In designing this dam, it was assumed that uplift pressures would act over one-third the area of the base and that the variation in pressure would be as a straight line running from full reservoir head at the up-stream side of the dam to full tail-water head at the down-stream side. Line 1, Fig. 20, shows the assumption of uplift-pressure variation, while Line 2 shows the equivalent line which would give similar uplift forces if applied to the entire area of the base. Line 3 shows the design assumptions used at the Willwood Dam, namely, that the pressure represented by Line 1 is applied over one-half the area of the base. Line 4 shows the uplift assumptions being used in the design of a high arch dam for an irrigation project in the Northwest, the pressures shown by this line being applied over the full area of the base.

FIG. 17.—GENERAL VIEW OF SPILLWAY SECTION, AMERICAN FALLS DAM, MINIDOKA PROJECT, IDAHO.

FIG. 18.—VIEW OF ROCKY FOUNDATION OF SPILLWAY SECTION, AMERICAN FALLS DAM.

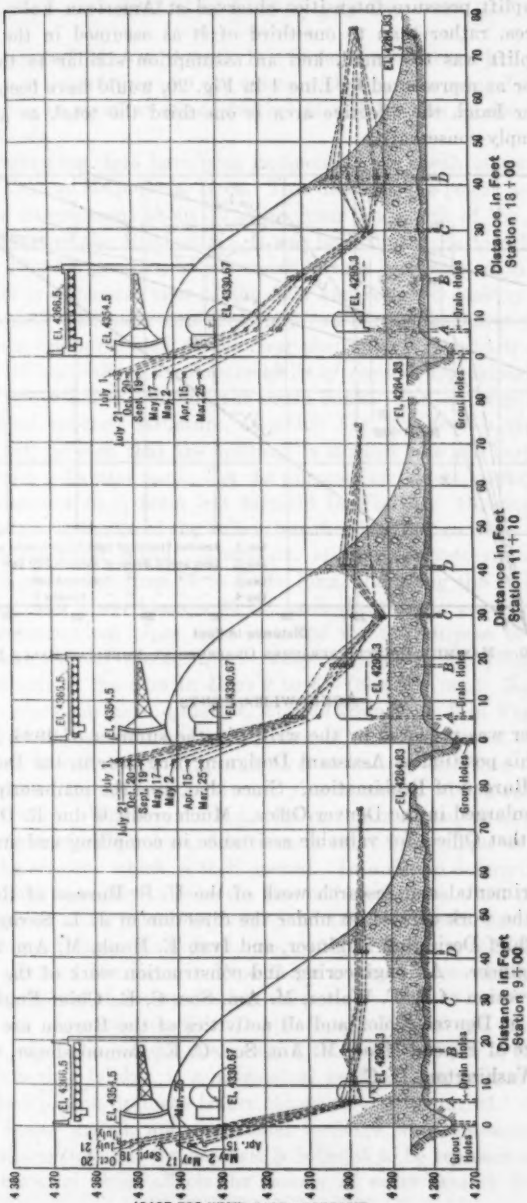


FIG. 19.—UPLIFT PRESSURES OBSERVED AT AMERICAN FALLS DAM.

If the uplift pressure intensities observed at American Falls apply to all the base area, rather than to one-third of it as assumed in the design, the assumed uplift was too small, and an assumption similar to that made at Willwood, or as represented by Line 4 in Fig. 20, would have been better. If, on the other hand, the effective area is one-third the total, as assumed, the design is amply conservative.

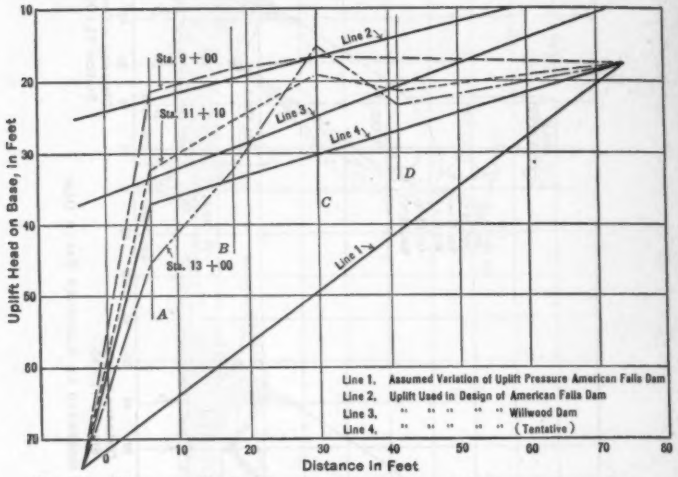


FIG. 20.—MAXIMUM UPLIFT PRESSURES OBSERVED AT AMERICAN FALLS DAM.

ACKNOWLEDGMENTS

This paper was prepared by the writer in the summer of 1926, just before he resigned his position as Assistant Designing Engineer in the Denver Office of the U. S. Bureau of Reclamation. Since that time the manuscript has been revised and enlarged in the Denver Office. Much credit is due R. D. Hubbard, Engineer of that Office, for valuable assistance in compiling and studying the data.

The experimental and research work of the U. S. Bureau of Reclamation is a part of the work carried on under the direction of J. L. Savage, M. Am. Soc. C. E., Chief Designing Engineer, and Ivan E. Houk, M. Am. Soc. C. E., Research Engineer. All engineering and construction work of the Bureau is under the direction of R. F. Walter, M. Am. Soc. C. E., Chief Engineer, with headquarters at Denver, Colo., and all activities of the Bureau are under the general charge of Elwood Mead, M. Am. Soc. C. E., Commissioner, with headquarters at Washington, D. C.

DISCUSSION

E. L. CHANDLER,* M. AM. SOC. C. E. (by letter).—The subject covered by this paper is very interesting and the writer wishes to offer some data from his own observations in an effort to add further material to the excellent contribution of Mr. Hinds.

Some enlightening data have been gathered as the result of work done on the Pinhook Dam at Maquoketa, Iowa. This dam and the power house form a hydro-electric development about 30 miles from the mouth of the Maquoketa River, a tributary of the Mississippi. It was built by the Eastern Iowa Power Company in 1923-24, ponding of the water having been started in December, 1923. Fig. 21 is a general view of the dam and reservoir looking up stream toward the west. The structure consists of an earth dike about 20 ft. high, extending from the north bank of the river about 400 ft. northerly to the hills on that side of the valley; a concrete spillway section approximately 160 ft. long; and a power house abutting the south bank. The spillway is a hollow deck, reinforced concrete structure, of which Fig. 22 shows a typical cross-section. It will be seen that the spillway is divided into six bays, each bay being fitted with a Taintor gate. For the purposes of this description, the bays will be numbered 1 to 6, from left to right in Fig. 21. Outcropping limestone is found on each side of the valley, but the river bed consists of sand and gravel to an unknown depth. Therefore, all the concrete structures were founded on timber piles from 20 to 30 ft. long, and along the up-stream line of the spillway and power house a 30-ft., steel, sheet-piling cut-off was driven.

During construction, pipes were installed for the purpose of measuring upward pressures under the spillway. They were placed in systems at right angles to the axis of the dam in Bays 2 to 6. (See Fig. 21.) Each set consisted of five pipes, 1½ in. in diameter, lettered from *A* to *E* in Fig. 23. Each pipe terminates at its lower end in a pocket of sand and gravel so graded as to permit easy flow of water. Pipes *A* to *D* extend upward into the chamber under the deck of the spillway, and each pipe is capped with a reducer and a ¼-in. brass air-cock. Whenever it is desired to obtain a reading at any station, a suitable length of rubber hose, fitted over a short ¼-in. nipple at one end, is coupled to the air-cock which is then opened. In order to determine the head acting, it is only necessary to raise the free end of the hose until the elevation is found at which water ceases to escape, and then, by measuring up from the known elevation of the concrete floor, it is a simple matter to reduce the results to a known datum. Readings in Pipe *E* are made by direct measurement in the pipe from the top of the tumble-bay weir.

The amount of percolation through any deposit of sand and gravel depends considerably, of course, on the character of the finer portions of the material. The tabulation that follows, is a mechanical analysis of a sample taken from the site of the Pinhook Spillway before construction was started. Considerable intermixed larger gravel, and occasional boulders were encountered in the course of construction, but the analysis is believed to be representative of that part of the material which affects the passage of water beneath the structure.

* With Price Bros. Co., Dayton, Ohio.

Size of screen.	Percentage passing.
1-in.	100.00
¾-in.	94.37
½-in.	89.83
⅜-in.	84.30
No. 4 mesh	71.21
No. 8 "	57.05
No. 14 "	39.93
No. 28 "	21.65
No. 48 "	2.68
No. 100 "	0.37

Readings were taken in the pipes at intervals during the filling of the reservoir and after the full head had been raised, the last one having been made in April, 1926. (See Fig. 23(*c*).) Results obtained only in Bays 3, 4, and 5 are considered herein, although other observations were made. Table 5 shows the results obtained. For the sake of comparison, the tabulation is made in a manner similar to that used by the author in Table 1. The nomenclature is somewhat different, but will be apparent from a study of Fig. 23. In studying Table 5, the first four groups of readings should be considered together. They represent conditions with wide variations of head, before the silt had accumulated over the bottom of the reservoir. The derived values for these groups, shown in Columns (10) to (19), inclusive, seem very consistent except for the values of the percolation factor, C_2. Variations for that factor are not surprising. Pressure readings were determined to the nearest 0.1 ft. only, and very slight errors in the values of h_2 would result in large variations in the corresponding values of C_2.

The fifth group represents conditions under which evidently there was a sealing of the bottom of the reservoir. The results show little relation between actual conditions and theory. In Bays 2 and 3 (Fig. 21), the pressures in Pipe *A* are actually less than the corresponding pressures in Pipes *B* and *C*. Quantities for Columns (15), (16), and (18), of Table 5, are not shown for this group as it is not considered that they would have any value under the circumstances.

For the purpose of making comparison between the pressures found and the theoretical values that would obtain if the "line-of-creep" theory were true, the curves of Fig. 23 are shown. These are based on readings given in Table 5 for Bay 4. It will be seen from this table that the results for this bay are typical. Computations for the theoretical piezometric line are based on the assumption of free communication between tail-water and the under side of the block apron, through the joint at the down-stream line of the tumble-bay weir. Fig. 23(*a*) and Fig. 23(*b*) represent conditions before any appreciable silting of the lake bottom had occurred, and would seem to offer excellent evidence in support of the "line-of-creep" theory under considerable variation of head. Fig. 23(*c*) is based on conditions existing more than two years after the completion of the dam. A mud blanket approximately 2 ft. deep had been deposited up stream. The very noticeable drop in the pressures for Pipes *A*, *B*, *C*, and *D*, as com-

FIG. 21.—VIEW OF PINHOOK DAM AND POWER HOUSE.

pared with previous readings for the same pipes, is perfect evidence of the value of an up-stream apron in reducing the upward pressure under such a structure.

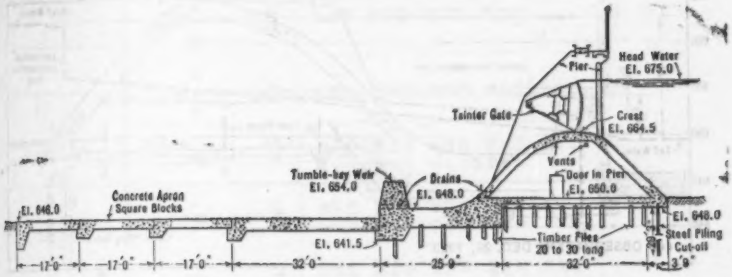


FIG. 22.—CROSS-SECTION OF PINHOOK DAM SPILLWAY, BAYS 2 TO 6.

It is found in each case that the inclination with the horizontal of the actual piezometric line is less than that of the theoretical line as computed, and that in Figs. 23(*a*) and (*b*), the two lines cross between the test pipes, *B* and *C*. A possible explanation for the fact that the pressures in Pipes *C*, *D*, and *E* are greater than those computed theoretically, might be that it is erroneous to assume that the theoretical line meets the tail-water elevation at the down-stream face of the tumble-bay weir. If it should be assumed that the concrete block apron confines the flow sufficiently to cause upward pressure under it, then it would be necessary to assume some point farther down stream at which the pressure would become zero. Such an assumption would lessen the amount of drop at the steel piling cut-off and would flatten the remaining part of the line, tending to make it approach the slope of the actual line.

A second assumption would be that, although the results afford a good substantiation of the theory, it is not strictly true that all the flow follows down the up-stream side of the steel piling and up the other side. It is probable that some of the flow diffuses through the pervious sub-strata, following a course leading diagonally upward from the bottom of the piling, and reaching the under side of the concrete farther down-stream. This would help to account for the comparatively lower pressures in Pipes *A* and *B*. Such would probably be the case in any pervious material of uniform structure, and would certainly be true if the piling extends through or into a stratum more open in nature than the overlying material.

An unfortunate feature of the Pinhook installation is the fact that no readings can be taken during a period of high tail-water because the water level under the spillway deck communicates freely with the tail-water through drain holes and when the tail-water rises it becomes impossible to reach the test pipes.

The author attributes the rather irregular nature of the results obtained from readings on the Colorado River Dam to the fact that while water enters beneath the dam through a considerable area up stream, it "can escape in appreciable quantity only through the narrow gap of open gravel under Pier

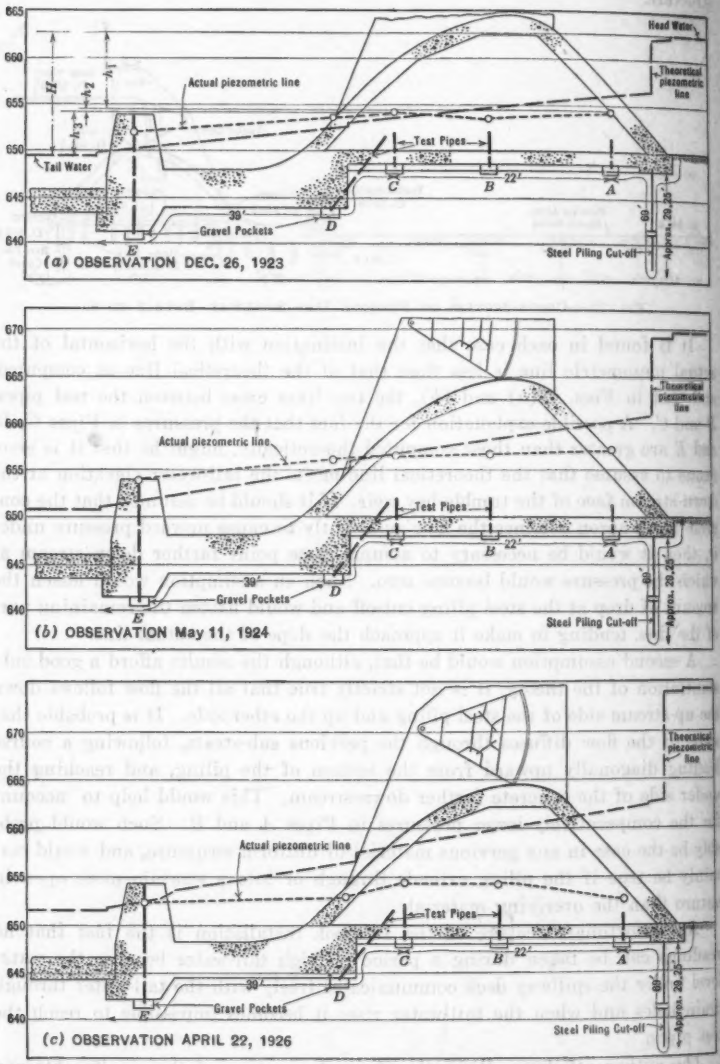


FIG. 23.—DIAGRAMS SHOWING COMPARISON OF ACTUAL AND THEORETICAL PIEZOMETRIC LINES.

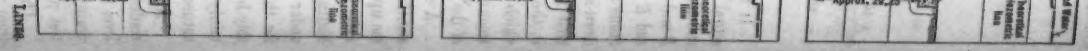


TABLE 5.—MEASURED UPLIFT UNDER PINHOOK DAM.

Date.	Bay.	Head-water.	*A.*	*B.*	*C.*	*D.*	*E.*	Tail-water.	h_1, Column (3)—Column (4).	h_2, Column (4)—Column (6).	h_3, Column (6)—Column (9).	H, Column (3)—Column (9).	$\frac{h_1}{H}$.	$\frac{h_2}{H}$.	$\frac{h_3}{H}$.	$\frac{C_1}{69 \div h_1}$	$\frac{C_2}{22 \div h_2}$	$\frac{C_3}{39 \div h_3}$
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)
12-26-23	3	662.7	654.8	654.3	654.2	652.2	652.6	649.4	7.9	0.6	4.8	13.3	0.59	0.05	0.36	9	37	8
	4‡	62.7	54.4	53.6	54.1	53.7	52.1	49.4	8.3	0.3	4.7	13.3	0.63	0.02	0.35	8	73	8
	5	62.7	54.3	53.3	53.8	*	51.5	49.4	8.4	0.5	4.4	13.3	0.63	0.04	0.33	8	44	9
1-10-24	3	66.3	57.0	*	56.2	55.8	*	49.6	9.3	0.8	6.6	16.7	0.56	0.05	0.39	8	28	6
	4	66.3	56.3	56.1	55.9	55.5	*	49.6	10.0	0.4	6.3	16.7	0.60	0.02	0.38	7	55	6
	5	66.3	56.3	*	55.7	*	*	49.6	10.0	0.6	6.1	16.7	0.60	0.04	0.36	7	37	6
2-11-24	3	72.0	60.5	58.9	58.7	*	*	51.2	11.5	1.8	7.5	20.8	0.55	0.09	0.36	6	12	5
	4	72.0	58.9	59.5	58.4	57.8	*	51.2	13.1	0.5	7.2	20.8	0.63	0.02	0.35	5	44	5
	5	72.0	58.8	58.6	58.1	*	*	51.2	13.2	0.7	6.9	20.8	0.63	0.04	0.33	5	31	6
5-11-24	3	69.8	57.3	57.1	57.3	*	54.0	50.6	12.5	0.0	6.7	19.2	0.65	0.00	0.35	6	8	6
	4§	69.8	57.1	56.8	56.6	56.1	54.0	50.6	13.0	0.5	6.0	19.2	0.68	0.02	0.31	5	44	6.5
	5	69.8	56.9	56.1	56.7	*	53.6	50.6	12.9	0.2	6.1	19.2	0.67	0.01	0.32	5	110	6
4-22-26	3	74.8	52.5	53.8	54.3	52.2	53.0	50.7	22.3	+1.8	3.6	24.1	0.93	†	†	3	†	11
	4‖	74.8	52.8	54.3	54.2	52.0	51.5	50.7	22.0	+1.4	3.5	24.1	0.92	†	†	3	†	11
	5	74.8	53.0	53.8	52.9	51.5	53.0	50.7	21.8	0.1	2.2	24.1	0.90	†	†	3	†	18

* Denotes readings were not obtained because of frozen pipes or other obstructions.

† Denotes quantities not computed because of reversed relations for h_2.

‡ See Fig. 23 (*a*).

§ See Fig. 23 (*b*).

‖ See Fig. 23 (*c*).

B", and that "after passing the restricted open gravel area under the cut-off, the flow is free to spread laterally, thus reducing the resistance to outflow". This is very probably the case. A fairly analogous condition was created artificially in a series of tests conducted by E. W. Lane, M. Am. Soc. C. E., and the writer, on the Island Park Dam at Dayton, Ohio.*

Island Park Dam is part of the park system of the City of Dayton. It is a gravity type, concrete, overflow structure, across the Miami River, developing a head of 7 ft. at low-water flow. Conditions are very similar to those shown in Fig. 9, except that the timber sheet-piling is at the up-stream line of the fore apron, and that one of the test pipes extends into the bed of the pond up stream. Tests were made after a mud blanket had sealed the bottom of the pond, and in every instance the readings were far below the theoretical pressures based on the "line-of-creep" theory. An attempt was then made to approximate conditions which existed before the silt deposit had formed. This was done by removing the mud from a small area in the vicinity of the up-stream pipe. As a result, the pressure in the up-stream pipe increased to nearly the full theoretical head, although there was slight change in any of the others. This was, no doubt, due to the diffusion of flow after passing through the comparatively small opening near the pipe intake.

The foregoing experiments indicate that, for the design of structures on sand and gravel foundations, it is not wise to assume pressures less than those that apply under the "line-of-creep" theory, although that theory probably errs somewhat on the side of safety. With the depositing of silt over the bottom of a reservoir, the factor of safety increases very materially. As the author indicates, in the case of a low structure, this margin of safety may be diminished in a time of high water if velocities are developed high enough to disturb the bottom up stream from the dam. In a case where no appreciable disturbance of the bottom will occur, it might be reasonable to assume less than the total amount of upward pressure, if the pond can be filled by easy stages and the bottom can be allowed time for partial sealing before the full head accumulates behind the dam. For economic reasons, however, it is generally desired to develop the full head as soon as possible, and such assumptions might lead to trouble.

Installations of apparatus of the nature described by the author can be accomplished at very little cost at the time of construction, and are advocated as serving two worthy purposes. The value of data obtained as a basis for theory and design is obvious. Furthermore, periodic observations will either give assurance of continued stability, or may indicate extraordinary conditions developing under a structure in time to avert serious damage or even failure.

B. F. Jakobsen,† M. Am. Soc. C. E. (by letter).—This paper is an interesting account of some measurements of uplift pressures under dams. A study of Tables 3 and 4 shows a progressive increase in uplift pressure

* *Engineering News-Record*, May 20, 1920.

† Cons. Engr. (La Rue & Jakobsen), Los Angeles, Calif.

intensities in nearly all the holes, except those nearest the down-stream face. This is very significant, and it appears that the water makes channels under the dam, progressing slowly until these channels finally reach the down-stream tail-water. Referring to Table 3 and comparing the values for May 2, 1923, with those of November 11, 1925, it will be noted that both the reservoir and tail-water elevations were lower on November 11, 1925, but most of the uplift pressures were higher, except in the holes, *D*, where these pressures have decreased and approached closer to the tail-water elevations.

It is also interesting to note in Fig. 19, that in spite of grouting holes spaced 5 ft. apart under the cut-off wall and in spite of the drain pipe, *A*, the uplift pressure intensities in Pipe *B* are considerably higher than at the tail-water (see especially Station 13 + 00). Reading Column (*D*) under Station 13 + 00 in Table 4 and comparing these readings with the water elevation in the reservoir and the tail-water elevation, the gradual increase followed by a decrease, may be explained by the fact that the water slowly penetrates under the dam until it finally reaches the tail-water.

It would have been desirable if several pipes could have been located in fairly close proximity to each other; say, in groups, so that some information might have been gathered as to how regularly the uplift pressure occurs and how it is distributed under the dam. In the absence of any specific knowledge it would seem to the writer that the measured uplift should be assumed as acting under the entire base, although this would be an upper limit that cannot be reached, but can be approached.

It would be interesting to have these measurements continued because it is possible that silting might occur, the water might scour larger and larger channels as time goes on, or uplift might increase at one section and decrease at another.

All that experience can teach is that uplift does occur. It will undoubtedly vary for each particular site and may vary from section to section at any site. It is quite possible that some uplift may occur even in good rock and in spite of such precautions as grouting under the cut-off wall and by drainage wells close to the wall.

If it were not for the possibility of uplift, the design of a gravity dam, simple as it is, might be classed almost as an exact science. In reality, the first step toward the design of such a dam, is to make an assumption as to how much uplift may be expected. If the base width of the gravity dam is less than about 85% of the height, some assumption as to uplift is involved. If the designer is too optimistic, an unsafe dam is likely to be the result.

Taken as a whole, the paper shows that uplift is a serious matter under gravity dams and that drainage and grouting cannot be relied on to eliminate it entirely.

CHARLES TERZAGHI,* M. AM. SOC. C. E. (by letter).—Data like those presented in Mr. Hinds' paper are among the most precious that the engineer could possibly obtain because they create a unique opportunity to check current conceptions against observed facts.

* Prof., Foundation Eng., Mass. Inst. Tech., Cambridge, Mass.

The first group of data concerns dams on a more or less permeable sand and gravel foundation. According to Bligh's empirical rule* a row of sheet-piles should increase the effective distance of percolation by an amount equal to twice the depth of the sheet-piles. During the past years, Bligh's rule has been also confirmed by both theory and laboratory experiment. Engineers should realize, however, that the theoretical and the empirical rule is only valid when the underground is perfectly homogeneous; that is, when the permeability of the underground is the same in a vertical and in a horizontal sense, and, in Nature, such simple conditions almost never exist. If the underground includes a single layer of feebly permeable material which is penetrated by the sheet-piles, the piling becomes far more effective than it would be otherwise.

Another important question concerns the processes which lead to piping. According to the results of the experimental and theoretical investigations performed by the writer a few years ago concerning the mechanics of piping in uniform sands, the material located beneath the structure remains strictly in equilibrium unless, at some point down stream from the weir, the hydraulic gradient exceeds the critical value, unity.† There is not even a shadow of a possibility for underground erosion. The structure either is safe or else it fails, but there is no possibility for the continued movement of particles. In contrast to this, wherever the underground consists of strata of widely different grain composition, every surface of contact between two different layers represents a zone of weakness. The seepage produced by storage starts an extensive migration of material within the underground. As a result of this migration, the quality of the underground gradually becomes either better or worse, but it certainly does not remain stationary and it may take many years until a state of relative equilibrium is reached. Tables 1 and 2 seem to furnish very instructive illustrations for these gradual changes. The results of experimental investigation of the migration of material in stratified underground were recently published by Dr. A. Läufer, in Vienna.‡ The factor of safety of a dam against piping depends in a large extent on minor geological circumstances which cannot possibly be anticipated, such as the area and form of lens-shaped beds of more permeable material. Therefore the gradual transformation of the underground on every dam should be watched for a couple of years by means of stand-pipe observations, such as those reported by Mr. Hinds. On some weirs in the Alps that were equipped with a foundation patented by the writer, provisions were made for checking or releasing the flow of seepage according to the results of stand-pipe observations after the weir was built.

Very uncertain, also, are the current assumptions concerning the hydrostatic uplift below masonry dams founded on solid rock. It should be realized that a solid rock foundation with fairly wide groutable fissures may be far less unfavorable than another one, equally solid, with many but very nar-

* "Dams, Barrages, and Weirs on Porous Foundations," by W. G. Bligh, *Engineering News*, 1910, II, p. 708.

† "Der Grundbruch an Stauwerken und seine Verhütung," by Charles Terzaghi, *Die Wasserkraft*, 1922, p. 445.

‡ "Lassenbildung und deren Verheilung," *Die Wasserwirtschaft*, 1927.

row fissures which cannot be grouted. Fig. 24 shows a cross-section through the Oestertal Dam, in Germany, and the distribution of the hydrostatic upward pressure as disclosed by measurements on May 6, 1910, at full reservoir.* The upward pressure was measured by means of pressure gauges connected through pipe lines with chambers 4 in. in diameter, arranged along the surface of contact between the dam and the rock. The pressures, AB (Fig. 24), were measured along a section where, during construction, the rock appeared to be tight and dry; while the pressures, AE, were measured along a section where the rock was somewhat scattered and contained conspicuous water-bearing seams. According to Fig. 24, along the first section, the upward pressure was almost twice as high as it was along the second one. The diagram shows, in addition, how extremely variable the distribution of pressures can be over the base of the same dam.

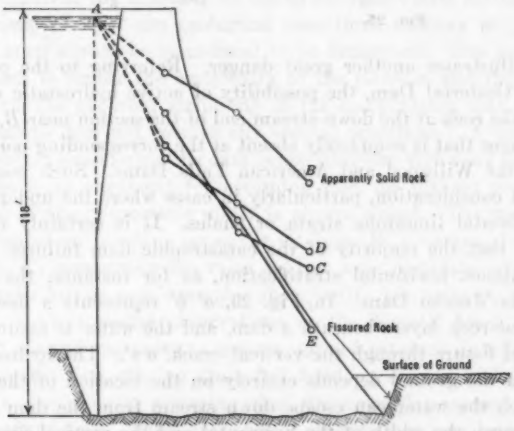


FIG. 24.—HYDROSTATIC UPWARD PRESSURE ACTING ON THE OESTERTAL DAM IN GERMANY.

Still more uncertain are the accepted assumptions concerning the percentage of the area of the base over which the pressure acts. The narrowest seams which can be grouted have a width of about $\frac{1}{20}$ in., while the narrowest seams through which hydrostatic pressure can still be transmitted, have a width of about $\frac{1}{500}$ to $\frac{1}{1000}$ in. Under the geological conditions found at the site of the American Falls Dam, in Idaho, it seems to the writer that the assumption of the pressure acting over one-third the area might have been fairly conservative. On the other hand, there are geological conditions which present the possibility that the pressure may act over more than four-fifths of the base area. This is particularly true in cases where the valley was carved out by erosion through rocks with a low modulus of elasticity. Due to the removal (by erosion) of the rock which formerly occupied the shaded area in Fig. 25, the remaining rock underwent unequal elastic expansion associated

* "Unterdruck bei Staumauern," by R. Schäfer, *Zeitschrift für Bauwesen*, 1913, p. 101.

with the formation of cracks and slight mutual displacement of the rock fragments. As a result of such a displacement, the area of actual contact between the rock fragments may be reduced to a small fraction of the area of the base of the dam, leaving between the rock faces both horizontal and inclined fissures too narrow to yield a visible amount of seepage, yet wide enough to transmit hydrostatic pressure.

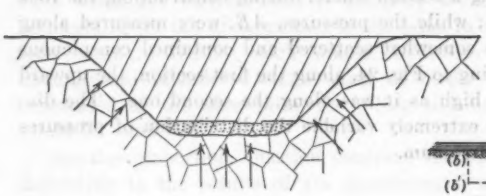


FIG. 25.

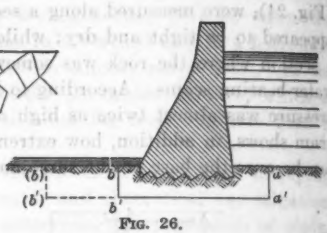


FIG. 26.

Fig. 24 illustrates another great danger. Referring to the pressure line, *AB*, for the Oestertal Dam, the possibility of active hydrostatic uplift occurring within the rock at the down-stream end of the section near *B*, is apparent. This is a danger that is completely absent at the corresponding points (near *E*, Fig. 24) at the Willwood and American Falls Dams. Such possibilities require careful consideration, particularly in cases where the underground consists of horizontal limestone strata or shales. It is certainly more than a mere chance that the majority of the catastrophic dam failures occurred on rocks with almost horizontal stratification, as for instance, the two Austin Dams and the Jumbo Dam. In Fig. 26, $a'\ b'$ represents a fissure between two horizontal rock layers beneath a dam, and the water is assumed to enter the horizontal fissure through the vertical crack, $a\ a'$. The hydrostatic uplift acting within the ground depends entirely on the location of the crack, $b\ b'$, through which the water can escape down stream from the dam and also on the ratio between the width of the horizontal and the vertical fissures. Since these important circumstances cannot possibly be investigated, either by inspection of the dam site, or by test borings, the design should be based on the most unfavorable possibilities. In a case like the one represented in Fig. 26, the greatest danger obviously consists, not in the action of the uplift on the base of the dam, but in the possibility that the upward pressure will lift the weaker roof of rock down stream from the dam.

Obviously, then, the intensity of the upward pressure acting on dams depends to a very large extent on the element of chance; that is, on circumstances which cannot possibly be predicted except in a very general way. The most dependable rocks are obviously those that contain a moderate amount of irregular groutable fissures, such as those that form the underground of the Willwood and the American Falls Dams. The grout curtain creates a considerable drop of the hydrostatic head at the place where it intersects the current of seepage water; while, down stream from the curtain, the open fissures facilitate the free escape of the seepage water toward the down-stream toe.

On the other hand, the most hazardous foundation conditions should be expected in almost undisturbed limestone and shale rocks that have very few vertical cracks. In such rocks, there exists, the additional danger that the horizontal fissures may be open over a large percentage of their area. This is due in part to the dissolving action of the seepage water and in part to the possibility of unequal expansion of the shale beds or of the gradual removal (by erosion) of soft material contained in the seams. To pass judgment on the quality of a dam foundation is one of the most difficult and responsible tasks. It requires both careful consideration of the geological conditions and the capacity for evaluating the hydraulic importance of the geological facts which can only be obtained by a thorough training in the hydraulics of seepage.

Many textbooks on engineering contain simple and alleged empirical rules for computing the intensity of the hydrostatic uplift acting on the base of dams, irrespective of the geological conditions existing at the dam site. Such rules must always be considered to be dangerous. The young engineer is led by them to ignore the complexity of the problem and the layman may conclude from the apparent simplicity of the computation that the design of dam foundations does not require any special expertness. A warning should be issued against considering the data concerning the Willwood and the Austin Dams as one more argument for assuming that the present practice for the computation of the hydrostatic uplift is always sound. The warnings which have been issued on previous occasions by John R. Freeman, Past-President, Am. Soc. C. E., concerning the same subject were certainly not exaggerated, and every year brings new examples that justify his opinion. Prompted by the results of his pressure observations at the Neyetal Dam, in Germany, R. Schäfer* advised engineers to assume upward pressure distribution according to a straight line, sloping from full head up stream to half head down stream. He recommends assuming that the upward pressure acts over the entire area of the base. Thus, every set of observations may lead to other conclusions. Nevertheless, if placed in the hands of cautious and experienced engineers, data such as those presented by Mr. Hinds are of inestimable value. In Europe, studying the uplift and the seepage conditions of dams continuously until they become stationary has become a fairly common practice; and it would be of great value if the same practice would become a routine custom in the United States.

J. C. Stevens,† M. Am. Soc. C. E. (by letter).—The data presented in this paper are invaluable and will aid greatly in the design of dams as regards uplift pressures. The simplified presentation of the maximum uplift pressures observed, compared with those used in design as in Figs. 15 and 20, is particularly to be commended. These show the pressure areas over the entire base of the dam. The writer believes this is the proper manner to treat such uplift pressures in design and also to present experimental data relative thereto.

* "Unterdruck bei Staumauern," *Zeitschrift für Bauwesen*, 1913, pp. 101-118.

† Cons. Hydr. Engr. (Stevens & Koon), Portland, Ore.

The author fails to describe the drain-holes in the American Falls Dam. Presumably, they are independent holes in line with the pressure pipes, *A*, but their spacing and depth are not indicated. Their effect, however, is quite apparent in Fig. 20. It is hardly possible that the pressure lines are straight from the *A*-line to the free-water surface, since this ignores the effect of the cut-off wall. Similarly, it is not likely that they are straight between rows of holes or between the *D*-line and tail-water. This shows that while unit uplift pressures may be measured at many points in the dam, engineers are yet almost entirely ignorant of the area to which these unit pressures apply.

It would seem that uplift in the mass of the concrete itself is possible, particularly along horizontal planes between pours and especially when the dam is loaded, so that there is practically no pressure in the masonry itself at the face of the dam.

If dams could be designed so that the pressure in the masonry at all points equals or exceeds the water pressure at that point, uplift would be taken care of automatically. This is impracticable in high dams, however, and uplift pressures must be reduced, first, by adequate cut-off walls and, second, by effective drainage down stream therefrom.

F. W. HANNA,* M. AM. SOC. C. E. (by letter).—There is probably no other question arising in the design of a dam, on which there is so much guesswork, as on the proper uplift to assume in order to meet actual conditions that occur in the foundation of the structure. This paper, therefore, is timely and valuable in the present period of extensive dam building. The fact that the U. S. Reclamation Service has provided facilities for observing pressures in three different types of foundation materials, is commendable, and additional similar facilities in other foundations are desirable.

The author's contention that filter bed action exists in the stream beds above overflow dams is undoubtedly correct, but it is of relatively small importance in the designs of dams made to withstand the worst uplift conditions. It is of extreme importance, however, when it comes to a proper interpretation of seepage data. The paper brings out this fact clearly. The erodability of the filter film, where the dam is relatively low, is well illustrated by the Colorado River and Percha Dams.

The author shows that the data at the Colorado River Dam neither prove nor disprove Bligh's theory of uniform pressure drop along the line of creep, and he properly ascribes this to the irregular foundation conditions at the dam. In this connection it is important to observe that conditions of such uniformity as to give results in conformity with Bligh's theory rarely occur in the foundations of dams. Of the four dams discussed by the author, only one has a fairly uniform foundation throughout. The Colorado River Dam is founded on loose, coarse gravel of varying thickness, with parts of the cut-off wall extending to tighter material. The foundation of the Willwood Dam is partly shale and partly sandstone; and the American Falls Dam rests on columnar basalt. In the foundations of all these dams, there is opportunity for lateral movement of the seepage water due to variations in the depth or character of the materials under them, thus destroying the conformity of

* Hydr. Engr., East Bay Municipal Utility Dist., Oakland, Calif.

pressure creep theory with the Bligh theory. On the other hand, the Percha Dam is founded on rather uniform material as regards horizontal layers parallel to the axis of the dam; and at this project there is a more reasonable conformity with Bligh's theory, as might be expected. The sandstone section of the Willwood foundation also shows reasonable conformity with Bligh's theory.

Another interesting point brought out by the data presented by the author is the distribution of pressure on the base of the dam. In the design of dams it is customary to assume the uplift pressure to act either on all, or on a fractional part, of the base. The lack of uniform distribution of pressure is well indicated by the points, *B*, in Lines 1 and 2 of the Willwood Dam and the points, *C*, at Stations 11+10 and 13+00 in the American Falls Dam. The foundation materials of these dams are such as to make these results subject to anticipation. On the other hand the foundation materials of the Percha Dam and the sandstone parts of the foundation of the Willwood Dam are such as to lead to the assumption of uniform distribution. In this connection the experiment* of H. de B. Parsons, M. Am. Soc. C. E., is of interest. In carefully conducted experiments, Mr. Parsons found, for confined clay, sand, and gravel, that the effective area of the base is approximately 100 per cent. This indicates that the pressure is applied to the base, not only by direct water contact, but by the points of contact of the soil particles with the base. This result is for uniform hydrostatic head on the entire base and is not to be confused with the variation of head from heel to toe of dam.

Ivan E. Houk,† M. Am. Soc. C. E. (by letter).—Some additional measurements and studies of upward pressures under the American Falls and Willwood Dams have been made since this paper was published. In view of the fact that one or two additional points of interest are exemplified, and also because it is desirable to furnish as complete data as possible for the use of interested engineers, it seems advisable to present the results of the more recent investigations in this discussion.

Table 6 gives the results of measurements made at the American Falls Dam since October, 1927, the arrangement of the data being the same as in Table 4. It will be noticed that observations have been made about once a month during the last year (1927-28). Present plans are to continue the readings at about the same interval for at least another year, and then at somewhat greater intervals for some time in the future. By such readings it is hoped to observe any increase or decrease in uplift pressures which may occur as time progresses. Thus far, no general conclusions as regards a time effect seem to be warranted.

Any thorough study of uplift pressure variations resulting from elapse of time should be based either on observations taken with the same reservoir and tail-water surface elevations, or on observations properly corrected for variations in total head. Variations in the total head at the American Falls Dam are essentially variations in reservoir levels, inasmuch as the tail-water elevation is kept practically constant in connection with the operation of the plant

* See p. 1317.

† Research Engr., U. S. Bureau of Reclamation, Denver, Colo.

TABLE 6.—MEASURED UPLIFT ON BASE OF AMERICAN FALLS DAM SINCE OCTOBER, 1927.*

(All elevations are between 4 280 and 4 360 ft. (see Fig. 19).)

Date of observation.	Reservoir water surface elevation, in feet.	Tail-water elevation, in feet.	Uplift Pressures Expressed as Equivalent Water Surface Elevation, in Feet.											
			Station 9 + 00.				Station 11 + 10.				Station 13 + 00.			
			A.	*B.*	*C.*	*D.*	*A.*	*B.*	*C.*	*D.*	*A.*	*B.*	*C.*	*D.*
November 21, 1927	54.43	88.6	96.44	93.04	91.84	91.24	09.14	97.26	97.17	90.89	17.15	07.64	94.25	97.26
December 30, 1927	50.90	96.50	99.43	97.42	97.14	96.44†	14.14	98.15	97.17	96.29†	19.15	07.14	94.30†	96.56
January 19, 1928	50.50	96.50	98.73	97.22	97.24	96.64	15.14	99.15	98.15	96.69	22.15	07.64	94.45†	97.16
February 18, 1928	49.05	96.50	99.23	97.22	97.23	95.34†	14.14	00.15	98.15	.....	23.15	08.14	.....	.....
March 19, 1928	47.48	95.30	99.48	.....	.....	96.84	13.64	00.15	.....	96.79	22.15	07.64	93.95†	97.16
April 19, 1928	46.78	96.50	99.73	.....	.....	94.84†	11.14	00.15	99.15	96.29	22.15	07.14	96.95	97.16
May 18, 1928	48.81	95.70	98.23	97.24	96.14	95.34†	07.64	97.65	97.17	94.89†	21.15	06.64	94.25†	97.16

* See Table 4 for results of earlier measurements.

† Below tail-water elevation.

of the Idaho Power Company about 600 ft. down stream from the dam. Keeping this in mind a cursory inspection of Tables 4 and 6 would indicate that for the same total head the elapse of time at the American Falls Dam has resulted in increased pressures at some of the pipes and in decreased pressures at others.

. Where uplift pressures are affected by both reservoir and tail-water levels, or, in other words, where the bottoms of the pipes are connected with percolation passageways leading to both up-stream and down-stream edges of the base, the increases in pressure may be caused by either a loosening of the passageways leading through the grouted rock to the up-stream edge of the base, or by a tightening of those leading to the down-stream edge of the apron or to the open drains. Likewise, decreases in uplift pressure may be caused by a tightening of the passageways above the wells or by a loosening of the ones below. Probably both actions have been occurring to some extent at the American Falls Dam. Approximate measurements of total flow of the drains have been made at times of observing uplift pressures, but the results are somewhat erratic and show no definite increase or decrease in drainage since the reservoir was first filled. Tightening of the percolation passageways may be caused by deposition of salts from the percolating water. Such an action is believed to be responsible for a reduction in drainage flow which has been observed during recent years at the Elephant Butte Dam on the Rio Grande Project of New Mexico and Texas.

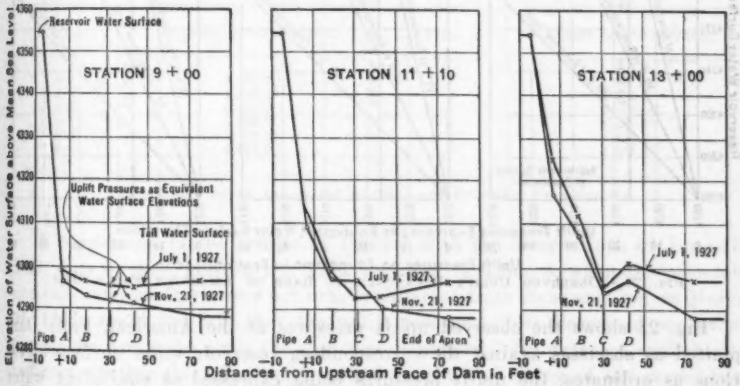


FIG. 27.—EFFECT OF VARIATION IN TAIL-WATER ELEVATION ON UPWARD PRESSURES UNDER AMERICAN FALLS DAM.

The series of uplift pressure observations at the American Falls Dam on November 21, 1927 (the first set of observations recorded in Table 6), is of special interest because of the relatively low tail-water elevation (4 286.6) which existed at that time. Such a stage is approximately 10 ft. lower than the elevation usually maintained. At the time of the November measurements the tail-water had been lowered for several days because of some gate repairs required by the Power Company. Fig. 27 shows the readings of November 21,

1927, platted with those of July 1, 1927, when the reservoir water surface was at almost exactly the same elevation, but when the tail-water level was at Elevation 4 297.0; that is, 10.4 ft. higher than on November 21. Water surface elevations are platted as ordinates, and distances from the up-stream face of the dam as abscissas—the down-stream direction being taken as positive. The diagrams show that the uplift pressures on November 21 were considerably lower than on July 1, the differences being from 3 to 4 ft. at several of the pipes and as much as 8.0 ft. at Pipe *A*, Station 13 + 00. (See Fig. 16.) The diagrams in Fig. 27, together with the data in Tables 4 and 6, would indicate that Pipes *C*, Stations 11 + 10 and 13 + 00, are relatively tight as regards connections with either reservoir or tail-water conditions, as stated by the author. No explanation can be offered for the sudden increase in pressure at Pipe *C*, Station 11 + 10, between July 21 and September 19, 1927. The other pipes, some of which have at times shown water elevations slightly lower than the tail-water levels, apparently are affected to some extent at least by both reservoir and tail-water conditions.

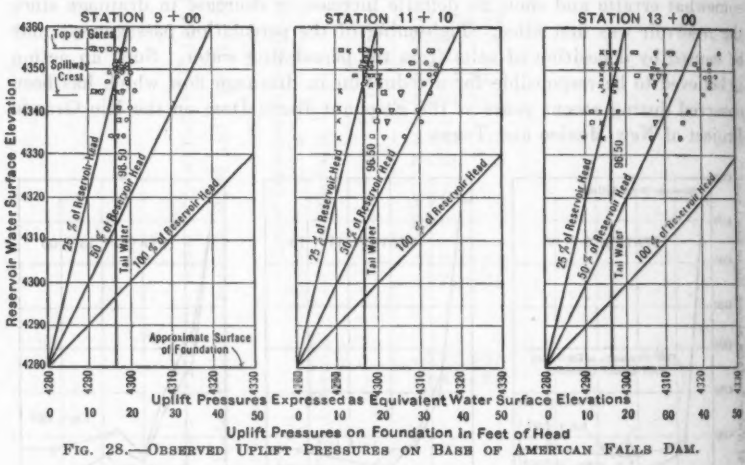


FIG. 28.—OBSERVED UPLIFT PRESSURES ON BASE OF AMERICAN FALLS DAM.

Fig. 28 shows the observed uplift pressures at the American Falls Dam platted as abscissas against the corresponding reservoir water surface elevations as ordinates, the uplift pressures being expressed as equivalent water surface elevations, as in Tables 4 and 6, no corrections being made for differences in tail-water levels. The elevation, 4 280, at the bottom of the diagrams is the approximate surface of the rock at the locations of the uplift pipes. Elevations of the spillway crest and the top of the radial gates are indicated on the diagrams by short horizontal lines, while the normal tail-water elevation, 4 296.5, is indicated by vertical lines. Diagonal lines represent uplift pressures of 25, 50, and 100% of the reservoir pressures at the up-stream edge of the base.

While the observations shown in Fig. 28 are naturally somewhat erratic, the diagrams indicate, in a general way, the variation of uplift pressures with

changes in elevation of reservoir water surface. It will be noticed that in most cases the uplift pressures fall between the 25 and 50% lines. Pressures at Pipe *A*, Station 13 + 00, are practically the only ones that fall definitely above the 50% line. These pressures fall on a diagonal line representing about 60% of full reservoir water pressure. Uplift pressures observed at Pipe *B*, Station 11 + 10, seemed to increase gradually as the reservoir filled until May 17, 1927, then to drop back about 6½ ft. by July 1, although the reservoir water surface continued to rise, reading an elevation of 4 354.46 by July 1—a total of 9.7 ft. higher than on May 17. Since July 1, 1927, the uplift pressure at this pipe seems to have remained almost constant, thus indicating that the foundation rock surrounding the bottom of the pipe has tightened appreciably. Observations at Pipe *A*, Station 9 + 00, seem to show a somewhat similar condition.

Measurements of uplift pressure at the Willwood Dam have not been made periodically for some time. In fact, only one set of observations has been secured since the paper was published. This was on March 31, 1928, when the reservoir water surface elevation was 4 500.55 and the tail-water elevation, 4 458.50. Uplift pressures at the various pipes on March 31, expressed as equivalent water surface elevations, are given in Table 7.

TABLE 7.—UPLIFT PRESSURES EXPRESSED AS EQUIVALENT WATER SURFACE ELEVATIONS.

Pipe.	Uplift, water surface.	Pipe.	Uplift, water surface.
A-1	4 467.2	*C*-1	4 461.3
A-2	4 458.4*	*C*-2	4 462.8
A-3	4 468.8	*C*-3	Plugged
B-1	4 452.9*	*D*-1	4 456.6*
B-2	4 457.7*	*D*-2	4 458.7
B-3	4 464.0	*D*-3	4 459.1

* Elevations below tail-water surface are indicated by asterisks.

A comparison of the results in Table 7 with the observations of June 16, 1923, July 6, 1923, and July 1, 1926, recorded in Table 3, when the reservoir and tail-water surfaces were not greatly different from those existing on March 31, 1928, would indicate that the uplift pressures have decreased somewhat during the five years since the dam was completed. No data are available to indicate whether this effect has resulted from a tightening of the percolation passageways above the pipes, from a loosening of the drainage passageways below, or from both.

It may be interesting to discuss briefly the methods of installing the uplift pressure pipes at the American Falls and Willwood Dams. At the latter the pipes ended at the line of contact between the rock and the concrete. The pipes were placed in holes drilled through the concrete after the body of the dam had been poured to Elevation 4 456.2, approximately 5 ft. above the surface of the rock foundation. It is presumed that the drill holes did not extend more than a few inches into the rock inasmuch as no mention of the matter

is made in the installation records. At the American Falls Dam the pipes were placed before concreting, leaving the lower ends 6 in. above the rock, as shown in Section *E-E* of Fig. 16. After the concrete was poured to a height of a few feet above bed-rock, holes were drilled through the concrete at the lower ends of the pipes and approximately 5 ft. into the foundation.

Whether or not holes should be drilled from the ends of the pipes into the foundation is a debatable question. If they should extend into the rock, how far should they go? If there are horizontal seams, or percolation passageways, in the foundation within relatively short distances of the base of the dam, the uplift pressures in such places may exert some uplift on the concrete, since the weight of the intervening rock would probably be considerably less than the uplift. Drilling into the rock may intercept such areas, or passageways, and result in the measurement of uplift pressures where otherwise no uplift would be recorded. At the Gibson Dam, which is a concrete arch type, 200 ft. high, now (1928) being built by the U. S. Bureau of Reclamation on the Sun River Project of Montana, the uplift pipes are being installed in the same manner as at the American Falls Dam, but the holes at the ends of the pipes are being drilled approximately 18 in. into the rock instead of 5 ft. The depth in the foundation to which such holes should be drilled, or whether they should be drilled at all, is a problem which is not susceptible of exact solution. It must be decided on the basis of engineering judgment. Of course, the pipes might be set directly on the rock, and the ends surrounded by gravel before concreting. However, such methods would tend to intercept uplift conditions from horizontal areas considerably larger than the areas of the pipes. In most cases the writer would favor drilling into the rock about 18 in., as is being done at the Gibson Dam. Probably the depth of drilling should be decreased for comparatively low dams and increased for comparatively high dams.

As regards the proportionate area of the base on which the uplift pressures should be considered to act when designing a dam, the writer is inclined to favor using the entire area, especially if the uplift pressure curve is based on actual measurements instead of on arbitrary assumptions. Thus far, no practicable method of measuring the areas on the base of a dam on which uplift pressures are actually exerted, has been developed. Naturally, the extent of such areas will depend on the character of the foundation, the thoroughness of the grouting operations, and the efficiency of the drains. If the dam is built on a solid rock foundation, the rock thoroughly grouted along the up-stream edge of the base, and an adequate system of drains installed, the writer would expect large areas of the base to be entirely free from uplift pressure. The existence of such areas at the Willwood and American Falls Dams is indicated by the zero uplift pressures observed at several of the pipes. However, if these pressures are taken into account in determining an average uplift pressure curve, no further consideration of such areas is necessary. As far as such areas are concerned the average pressure curve should be applied to the full area of the base. If estimates of the actual areas subjected to uplift pressure are to be made and used in calculating the total uplift force, the observations which indicate zero pressure should be omitted from the diagrams used in determining the intensity of the uplift pressure.

Of course, there remains the question as to what proportion of the rock surface is in contact with the concrete in the areas where uplift pressure is observed; or, more exactly, what proportion of the areas where uplift pressure is observed should be considered as capable of transmitting uplift pressure to the base of the dam? Here, again, the writer would favor using the entire area. If a crack has formed between the foundation and the dam, as was the case at the up-stream side of the Stevenson Creek Test Dam, the full area would be effective as far as the crack extended. If the rock is of a stratified nature and a horizontal seam exists a short distance below the base, uplift pressure may be transmitted through the rock layer to the full area of the concrete even if a large proportion of the rock surface is in actual contact with the concrete. If the dam has been built on a sandstone foundation and uplift pressure exists in the sandstone, the pressure may be effective over nearly the full area of the base even if a large proportion of the sand grains are in actual contact with the concrete. In such a case the pressures would be transmitted through the sand grains from pore spaces below. In this connection it is interesting to note that H. de B. Parsons, M. Am. Soc. C. E., as the results of a large number of experiments made under varying conditions of head, depth of sand, etc., concluded* that:

"* * * it would not be safe to estimate an uplift in pervious soil of less than full pressure head acting on the total area of the base, when there is no flow of water through the soil; nor less than the full pressure head indicated by the hydraulic gradient when there is flow through the soil."

The writer's idea of the most practicable method of investigating upward pressure under dams would be to install a large number of measuring pipes, uniformly distributed over the entire area of the base. He would then plat the pressure measurements, including the observations of zero pressure, and would apply the resulting intensity curves to the full area. Such a method would make proper allowances for the areas in which no uplift pressure exists. As regards the areas subjected to uplift pressure it would be slightly on the safe side. The pressure intensities would be accurately measured, but the effective area would be assumed as equal to the maximum possible. However, the writer believes that the question of effective area where there is uplift pressure is not so important as the question of what areas are entirely free from uplift. In other words, he believes that it is important at this time to secure a large number of uplift pressure measurements uniformly distributed over the base of the dam. When such investigations have been made at a large number of dams, built on different types of foundation, the designing engineer can make a much better estimate of the uplift to be expected at a given site than is possible at present.

C. H. HOWELL,† M. AM. SOC. C. E. (by letter).—This paper gives additional information on the intensity of uplift under certain existing dams. The effect of uplift is due to two factors: One of which is the intensity of the pressure; and the other, the area of the base on which the pressure is applied.

* See p. 1331.

† Chf. Engr., The J. G. White Eng. Corporation, S. en C., City of Mexico, Mexico.

As the author states the observations recorded do not prove or disprove the theory and assumptions ordinarily used in considering the problem. The fact that dams that have been investigated continue to stand, however, proves that the assumptions made in their designs were adequate.

The writer knows of no experiments or investigations as to the area subjected to uplift on rock foundations. This factor is just as important as that of the intensity. Its exact determination would be difficult if not impossible for the whole of any given dam site. Even if exact measurements could be obtained for one site the results would not necessarily be applicable to any other.

Water may develop pressure on the bottom area of a dam in one or both of two ways. One of these is through a faulty contact between the concrete and the top of the rock. The area thus affected must always be less than the total area as shown in Fig. 29(*a*). This is true, because otherwise there would be no contact between the concrete and the foundation, which would be an impossibility. The section, $C\,L$, is the part of the unit length, L, of the base receiving uplift from faulty contact. The factor, C, may vary from zero to anything less than 1.

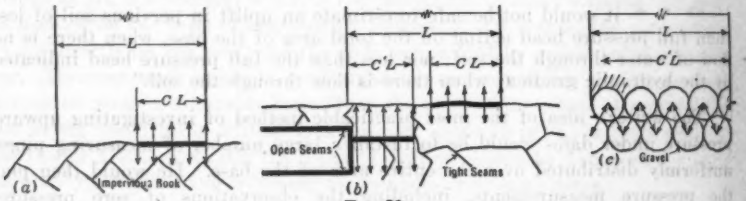


FIG. 29.

The other way for uplift to be developed is through open seams in the rock below the base of the dam (see Fig. 29(*b*)). The area thus affected may easily be 100% of any given unit of area; that is, the factor, C' (in Fig. 29(*b*)) may vary from 0 to 1.0. Likewise, $C'\,L$ is the section of unit length, L, of the base receiving uplift through open seams. It seems to the writer, however, that it is unreasonably conservative to assume 100% over the entire base of the whole dam.

Fig. 29(*a*) and Fig. 29(*b*) also illustrates the difficulty of making a quantitative determination of this part of the problem. In addition to the separate action of these two conditions the uplifting force may be the result of a combination of the two. The area subject to uplift on gravelly foundations has been shown by the author to be approximately 100 per cent. Fig. 29(*c*) shows this condition digrammatically. In this case, C' may vary from 0.9 to 1.0.

Until the areas affected by uplift can be determined the treatment of the problem on rock or similar foundations will continue to be a matter of judgment rather than mathematics. The multiplication of intensity observations alone will not serve to solve it.

P. WILHELM WERNER,* ASSOC. M. AM. SOC. C. E. (by letter).—The measurements at the Colorado River Dam, although indefinite, are of a very interesting nature. The author attributes most of the irregular character of the results obtained to a variable seasoning of the filter surface. For this special case, the explanation may be exactly true, or, at least, nearly so. The writer believes, however, that in many cases the "time effect" accounts for the observed irregularities in the pressure line to an appreciable extent. A certain retardation will occur in the formation of the state of equilibrium corresponding to a new head or differential pressure.

In general, the time effect is, evidently, a condition that is rather favorable to the stability of a dam, and the writer is of the opinion that it often adds materially to the safety of the structure. This is especially true if the flood peaks are of comparatively short duration and the "retardation coefficient" of the foundation is high. The time effect is of a universal nature, and applies to soils as well as rock strata where the water is creeping in very narrow channels. Sufficient data are not yet available, however, to accomplish a general study of its character and magnitude for different materials. It is to be hoped that the problem will soon be taken up in full and given the attention it deserves among engineers.

The readings at the Willwood Dam show that uplift under dams on rock foundation is a really serious matter and that extreme care must be exercised in selecting the dam site with regard to the actual geological and hydrogeological formations. If the shale and the sandstone in this case were located in the reverse order, there would probably have been a considerably more unfavorable condition of uplift under certain parts of the dam. When a dam is to be built on a rock foundation composed of very different strata, there is every reason to test the foundation hydraulically in order to prevent any unexpected eventuality.

For several reasons it would be of great value to have these experiments made over a rather long period of time. It would be especially interesting to know how the uplift pressure varies with the distance to the vertical construction and contraction joints. The results at the Willwood Dam seem to give some indication of this, but sufficient data are not available to justify a conclusion.

The writer believes that a vertical contraction joint, at least when it opens up, acts somewhat as a drainage well, thus decreasing the uplift under the base of the dam. The necessary requirement is, of course, that the joint shall not become clogged and that the seepage can freely issue into the tail-water. As contraction joints in solid gravity dams are usually constructed, however, it seems rather doubtful whether it is safe to assume an effective drainage, although it is more than probable that in many cases such joints have had an appreciable effect on the stability of dams. The writer is of the opinion that this question involves an economic factor of importance, and, therefore (assuming that there are favorable conditions at the dam site), he would suggest a solid gravity dam as shown in Fig. 30.

* Stockholm, Sweden.

This dam would be built in sections of ordinary length. The blocks would be separated by permanent contraction joints, extending down to rock, which would be made wide enough to permit the removal of the forms. This arrangement would cause an increased expense for shutterings, but would mean a considerable saving in concrete due to the decrease in uplift. Preliminary estimates indicate that for dams of ordinary size a saving of at least 5% of the total cost is quite feasible even under the assumption that the value of the ratio between the unit prices of shutterings and concrete is unfavorable.

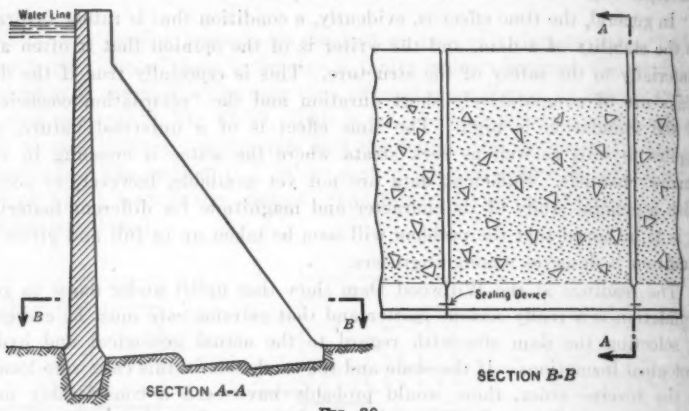


FIG. 30.

The writer is well aware of the fact that, in certain respects, this suggestion is somewhat against present common practice, and a discussion would probably involve many of the most important questions relating to the design of solid gravity dams. Some of these points are in distinct favor of the new design; that is, the possibility of a convenient inspection of the joints and the facilitated dissipation of the chemical heat in the masonry. Other points seem to be more or less a matter of opinion; for example, the hypothetical mutual support between different parts of a solid gravity dam as a means of preventing ultimate failure. Whatever the judgment may be, the writer thinks the idea is worth considering.

H. DE B. PARSONS,* M. AM. SOC. C. E. (by letter).—As knowledge regarding hydrostatic uplift pressure under dams is meager, all records of actual pressures are welcome. The profession is indebted to the U. S. Bureau of Reclamation for making the experiments reported by the author.

There is an upward pressure on the base of every structure founded below water level on fissured or pervious material. The intensity of this pressure undoubtedly would vary with differences in foundation material—its granular structure, grading of particles, fineness or coarseness, compactness, solidity, etc.

* Prof. Emeritus, Rensselaer Polytechnic Inst.; Cons. Engr., New York, N. Y.

Because all the factors involved are not appraised, it is to be expected that measurements of upward pressures will often appear inconsistent, that is, will apparently lack proper relationship. The water column in a test pipe only records static head. One pipe of a series may enter a firmly compacted pocket through which there is little or no flow, while another pipe in the same series may end in loosely compacted and coarse material, or in a crevice in the rock through which there is considerable and rapid flow. The water column in the first pipe would measure practically total head, while that in the latter would record only the static head. When plotted, the points representing the upward pressures, as measured in these pipes, would lack correlation with each other.

The work of preparing the site for a dam and the laying of the base masonry disturb Nature's placement of the foundation material. Water flowing through the foundation material will seek lanes or paths of least resistance, and these lanes may be neither straight nor at the surfaces of the foundation under the base. Consequently, observations of upward pressures in pipes arranged in a row normal to the axis of the dam may not show consistent relations, nor be such as to indicate a smooth hydraulic grade line. Furthermore, cut-offs will affect the uplift observations and produce results that may appear erratic.

Cut-offs made of sheet-piles may not be water-tight. The piling might be tight for some distance and then have an opening caused by a boulder or other sub-surface obstruction. The path of the water passing through such an opening cannot be predicted.

Deposit of silt will tend to close the voids between the soil particles, and thereby diminish the flow through the foundation. This silting process may divert the flow from some observation pipes and not from others. The silt blanket in a reservoir behind a dam, the crest of which is high above the river bed, will not be disturbed ordinarily by flood discharges, as mentioned by the author for the Colorado River Dam the crest of which was only 8 ft. high.

Considering the complexity that exists, the writer does not think that the quotients obtained by dividing the distances, as measured along the base of a structure in the direction of river flow, by their corresponding losses of static head (the percolation factors of Bligh), should be proportional in every instance. While the losses of static head may be measured with fair accuracy by the test pipes, the lengths of the paths along which the underground water flows is not known with any certainty.

The hydrostatic head in the pipes will be affected by the distance the perforations or pipe ends are below the base of a dam. Thus, if one pipe is deep and another shallow, the hydrostatic heads may lack correlation. It is the writer's opinion that, under ordinary conditions, the pipe openings should be not less than 18 in. and not more than 6 ft. below the base. This would place the openings of the pipes below the irregularities of the base masonry and still near enough to record the uplift pressures in the layer of foundation material on which the base rests. The chance for consistent and correlating records will be improved by care in placing the pipes.

The writer's plans for the Sherman Island Dam*, across the Hudson River, near Glens Falls, N. Y., contemplated thirteen rows of three pipes each; that

* *Transactions*, Am. Soc. C. E., Vol. 88 (1925), Fig. 18, p. 1287.

is, one row in alternate bays of the multiple arches, or about 38 ft. apart as measured across the river. Unfortunately, as constructed, only seventeen pipes were placed, with four bays having two pipes, and three bays, three pipes, each. In one bay there was a defective pipe. All of them were on the down-stream side of the 55-ft. interlocking steel sheet-pile cut-off. The foundation material under the dam was sand of varying sizes, containing some boulders. The geologic rock floor, except at the abutments, was not reached by 80-ft. borings.

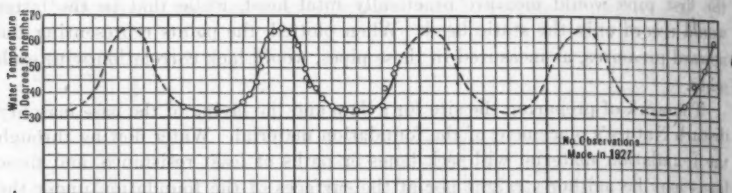


FIG. 31.—SEASONAL VARIATION OF UPLIFT HEAD UNDER MAIN DAM AND WATER TEMPERATURES, SHERMAN ISLAND DAM.

The upward pressures were fairly consistent in the pipes of each row, but were not always correlated when comparing one row with another. The uplift records showed an annual periodic variation which fluctuated with the changes in temperature of the river water. Thus, the average uplift head above water below the dam became greater as the temperature of the water increased and less as the temperature lowered. This seasonal variation is shown in Fig. 31. It was noticed that the up-stream cut-off of sheet-piling caused a large diminution in uplift, after which the uplift diminished toward the down-stream edge of the apron along a curved line. The low uplifts for 1925 may be due to low-water temperatures, as no temperature observations were recorded in that year during the summer months.

E. W. LANE,* M. AM. SOC. C. E. (by letter).—The data on upward pressure under dams presented by Mr. Hinds is a valuable addition to the present knowledge of this subject. The writer believes strongly in the necessity of such experiments because unsupported theoretical deductions are apt to be considerably in error.

He has collected some similar data which may throw additional "light on the subject". With E. L. Chandler, M. Am. Soc. C. E., he measured the upward pressure on the Island Park Dam in the Miami River at Dayton, Ohio. In these tests the effect of the blanket of silt which formed in the pool

* Engr., U. S. Bureau of Reclamation, Denver, Colo.

above the dam was especially noticeable, as it cut off nearly all upward pressure, as shown in Fig. 32 (*a*) and Fig. 32 (*b*). Another example of very slight upward pressure, probably due to the same cause, is that on the dam in the Miami River at Hamilton, Ohio. This dam is founded on gravel of good grading between sand and 9-in. cobblestones. The pipes were installed and the closure made in November, 1921, and the readings were taken in the following July by C. H. Schwartz, Assoc. M. Am. Soc. C. E. The results are shown on Fig. 32 (*c*) and Fig. 32 (*d*). Between the time of the closure of the dam and the date on which the readings were taken, several freshets occurred, which deposited a layer of mud above the dam. About 100 to 200 ft. above the dam, however, a gravel plant had excavated a hole 20 to 30 ft. deep.

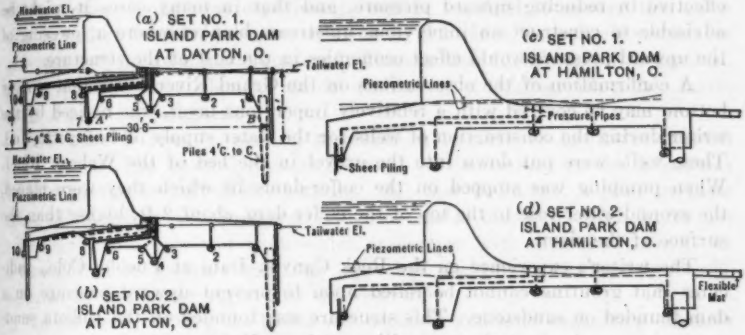


FIG. 32.—OBSERVATIONS ON UPWARD PRESSURE.

Another case showing upward pressure which was probably somewhat reduced by silting is that of the hydro-electric plant at Hamilton, Ohio (Fig. 33). The observations were made by Mr. Schwartz and Mr. W. P. Watson, three months after the water was turned into the head-race. The

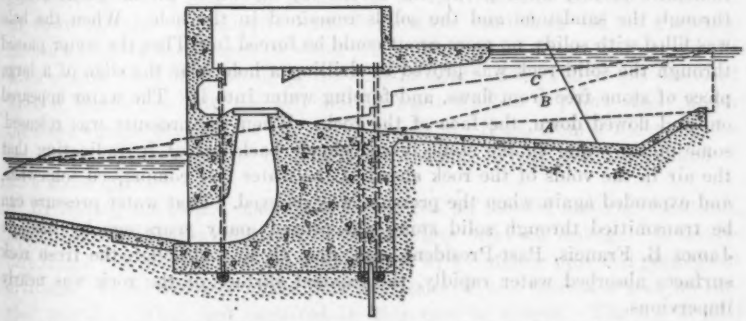


FIG. 33.—UPWARD PRESSURE EXPERIMENTS, HYDRO-ELECTRIC PLANT, AT HAMILTON, OHIO.

dotted line shows the upward pressure under the main part of the structure. The extension (*B*) of this line represents the assumption that the pressure at the up-stream edge of the structure is equal to the full depth of water at that point. Another assumption, represented by Line (*A*), is that the rate of

loss of pressure per foot of length of line of creep was the same under the up-stream part of the structure as under the part where it was established by the observations, and that the difference between Lines (*A*) and (*B*) at the up-stream edge represents the loss of pressure due to silting. The true condition is probably between the two assumptions. Since the length of the power house was relatively small in proportion to the effective head, it is probable that some of the water that entered the ground from the head-race passed around the ends of the power house and thus lowered the pressure more than it would have if all the water had passed under it. The dashed line (*C*) shows the upward pressure under the commonly used "line-of-creep" theory.

These examples show that an earth blanket up stream from a dam is very effective in reducing upward pressure, and that in many cases it might be advisable to construct an impervious up-stream blanket where a lowering of the upward pressure would effect economies in the cost of the structure.

A confirmation of the observations on the Grand River Dam, that a river bottom may be covered with a relatively impervious layer, was noticed by the writer during the construction of wells for the water supply of Lafayette, Ind. These wells were put down into the gravel in the bed of the Wabash River. When pumping was stopped on the coffer-dams in which they were placed, the ground-water rose to the top of the coffer-dam, about 2 ft. higher than the surface of the river.

The writer's experience on the Rock Canyon Dam at Pueblo, Colo., indicates that grouting cannot be relied upon to prevent upward pressure on a dam founded on sandstone. This structure was founded on the Dakota sandstone, composed of nearly pure silica grains. Holes on 10-ft. centers were drilled near the up-stream face of the dam into the rock to an average depth of about 10 ft. and were vented to the down-stream side of the dam to relieve upward pressure. The possibility of grouting these holes was investigated, but it was found that, although water could be poured into them slowly for an indefinite period, when grout was forced in, the water in the grout filtered through the sandstone and the solids remained in the hole. When the hole was filled with solids, no more grout could be forced in. That the water passed through the solid rock was proved by drilling a hole near the edge of a large piece of stone free from flaws, and forcing water into it. The water appeared on, and flowed down, the face of the rock. When the pressure was released, some of the water in the stone flowed slowly back out of it, indicating that the air in the voids of the rock ahead of the water was compressed somewhat, and expanded again when the pressure was relieved. That water pressure can be transmitted through solid stone was proved many years ago by the late James B. Francis, Past-President, Am. Soc. C. E. Although the fresh rock surfaces absorbed water rapidly, the exposed surface of the rock was nearly impervious.

C. E. GRUNSKY,* PAST-PRESIDENT, AM. SOC. C. E. (by letter).—The results of such studies as presented in this paper are of unquestioned value to the Engineering Profession. The same may be said of the great mass of research work and refined studies now in progress throughout the country for the ascer-

* Cons. Engr. (C. E. Grunsky Co.), San Francisco, Calif.

tainment of stresses in structural members and shapes under all manner of loading. It should not be forgotten, however, that actual conditions at completed structures cannot always be reproduced in the laboratory and that in the design of important works much relating to adequacy must frequently be left to personal judgment based on knowledge of past experience.

Thus, for example, the writer holds that experience has not yet demonstrated the effect on the quality and serviceability of concrete through which there is percolation of moisture in sufficient quantity to leach out carbonate of lime. The resulting efflorescence or deposit is frequently seen on the under surfaces of short-radius, multiple-arch concrete dams. Does concrete through which moisture is penetrating under high pressure lose strength? and, if so, at what rate? Long-time experiments will be required to answer this question as well as further questions relating to the effect in such circumstances of waters varying in mineral content. The engineer in this case is compelled, for the present, to supplement his determination of strains with a fair increment of personal judgment.

The same is true in making assumptions relating to hydrostatic uplift under dams. The upward pressures cannot be determined in the laboratory for any particular full-sized structure. All that is known, is that in those spots of uncertain and indeterminate area under dams to which water has relatively free access and from which it escapes with difficulty there is an accumulation of pressure that may approach the full hydrostatic head represented by the water surface of the reservoir.

To illustrate, in 1892 the writer was engineer and later consulting engineer on a California water supply project which involved the erection of an earth dam. It was built on a foundation of fissured volcanic tufa. The earth available for construction could not be called first-class. It lacked homogeneity. The earth fill extended from the dam for some distance along the top of a low ridge. Soon after the reservoir was filled, a part of this low extension where it had a height of perhaps 10 or 12 ft. became soft and sloughed off down stream. Familiarity with foundation conditions led to a speedy determination of the source of water which had caused the saturation of the down-stream third of a section of the embankment. This source was disclosed when a trench cut into the soft sliding mass reached a point about under the down-stream edge of the flat top of the dam. Here, from a fissure in the bed-rock a minute quantity of water was flowing. There was not enough to fill a goose quill.

By leaving the trench open for a day or two all the wet material dried out, and it became apparent that there was no other bed-rock leakage. A miniature trench cut into the bed-rock and covered with concrete provided permanent drainage. Then, the slope of the dam was restored, and there has been no further trouble. What had occurred in this case is simple. The fill at and near its toe had been well compacted, was fairly impervious, and in good contact with the foundation. Consequently, the water issuing from the fissure acquiring practically full hydrostatic pressure rose upward in the earth fill and gradually saturated a considerable mass of the down-stream slope and caused the slide.

Sound personal judgment is of great importance when the sufficiency of a foundation for a storage dam is to be ascertained. Moreover, there will nearly always be much doubt as to the distribution and magnitude of the upward pressure under storage dams and no generally applicable law can be laid down. The upward pressure is not necessarily confined to the joint between the structure and its supporting foundation, nor to horizontal cracks, but may find greatest development at some depth below the base of the structure.

The safe procedure does not lie in assumptions as to its magnitude, but rather in thorough drainage, not only of the material of which a massive dam is constructed, but also of the rock or other material on which it rests. Thus, for example, in the case of the earth dam of 1892 already referred to, a masonry wall was constructed across the creek channel about under the center line of the dam and a sump of broken rock was provided at the up-stream face of this wall, from which a pipe in a hillside trench led off the intercepted water. Months after the reservoir came into use, the discharge from this pipe was 186 drops per min. This is a very small quantity of water and yet, in the course of time, it might have saturated quite a mass of material within the dam if an outlet for it had not been provided.

JULIAN HINDS,* M. AM. SOC. C. E. (by letter).—It is gratifying to note the volume of useful material that the discussion of this paper has brought forth; it will be of great help in the solution of uplift problems. The constructive and helpful spirit of all the discussors is appreciated. The discussions are clear and complete and require few closing comments.

Mr. Chandler submits a complete and interesting set of data for the Pinhook Dam, which is founded entirely on gravel. These data indicate Bligh's "line-of-creep" theory to be safe for this structure. The blanketing effect of a mud deposit over the river bed, as found at the Percha and Grand Valley Dams, is again encountered here.

Mr. Jakobsen calls particular attention to the evidence that grouting and drainage do not entirely relieve uplift pressures in the case of rock foundations. This fact is believed to be generally recognized, but it is worthy of further emphasis, as a caution to those who may have too great confidence in the effectiveness of grouting.

The discussion presented by Professor Terzaghi is evidently the result of a very thorough comprehension of the status of the uplift problem. It amplifies the original paper, explains some of the observed phenomena and cautions against overconfident use of the data. It is noted that the uplift shown (Fig. 24) for the Oestertal Dam is high and drops approximately along a straight line. The tail-water level is not shown. Apparently, this dam is not grouted.

Mr. Stevens notes that the drainage provisions at American Falls are not properly described. Unfortunately, the writer does not have at hand data showing the spacing and depth of the drain holes as actually drilled. The construction drawings specify holes on 5-ft. centers, or as required by the engineer.

* Care, Dept. of Water and Power, Los Angeles, Calif.

Mr. Hanna agrees in general with the analysis of the original data, but joins some of the others in pointing out the improbability of finding exact agreement between Bligh's theory and actual pressures, because of lack of homogeneity in the underlying strata.

Mr. Houk brings the original data up to date by submitting subsequent measurements at the Willwood and American Falls Dams. He also adds much to the theoretical discussion. He recommends more closely spaced pipes in order to obtain an idea of the extent of the area subject to uplift which is believed to be good.

Mr. Howell is inclined to the theory that even in areas known to be subject to uplift on rock foundations, the presence of a 100% pressure area is impossible except in small pockets, because there must be some water-tight contact between the dam and the foundations. His discussion serves as a check on those who may be inclined to favor the assumption of a 100% effective area. It seems reasonable to assume that when one solid body rests upon another there must be some parts of the adjacent areas in water-tight contact, but until more information is available on the subject it is believed that the burden of proving that such areas necessarily constitute an important percentage of the total area must fall on those advocating the theory.

Mr. Werner suggests the possibility of time effects on the measured values of upward pressure. Undoubtedly, such effects exist, but the data at hand are not sufficient for their determination. It is, perhaps, unnecessary that there shall always be a marked lag in pressure changes with variations in head, after all the pressure cavities have been filled with water. Attention is called to the possibility that a change of strata, as at the Willwood Dam, may occur in such a way as to increase materially the uplift pressure. For example, a dam founded generally on a porous rock, but with a strata of tight rock along its down-stream toe, would probably be subjected to high uplift pressure. Such a condition should be avoided.

Mr. Werner's suggestion as to the relief of uplift* by means of wide open contraction joints, as illustrated in Fig. 30, is of particular interest to the writer because of certain supplementary studies made along this line. The security of many of the old Spanish dams found in Mexico is believed to be due to this principle. In these dams the separation is carried further than indicated in Fig. 30, the dam becoming a wall supported by heavy buttresses. By inclining the up-stream face, and with conservatively thick buttresses and walls, a theoretical saving of much more than the 5% suggested by Mr. Werner can be made with this type of construction.

Mr. Parsons emphasizes a number of reasons why a given set of uplift observations cannot be expected to agree within themselves, or to show exact conformity with any given theory of computation. This part of the discussion is followed by the presentation of data showing the effect of water temperature changes on the uplift at Sherman Island Dam. It is surprising to

* This principle of uplift relief, combined with other advantages and disadvantages, reaches its ultimate development for a gravity type of structure, in Noetzli's "Round Head Buttress Dam," as described in the latest edition of "Design and Construction of Masonry Dams," by Edward Wegmann, M. Am. Soc. C. E.

note that almost 100% increase in pressure is shown from the lowest to the highest temperatures.

Mr. Lane submits valuable new data and discusses, in some detail, the effectiveness of a silt blanket in the stream bed for reducing the uplift under a dam founded on gravel. The possibility of taking advantage of such an apron in design is suggested. Such a procedure is practicable in places where the blanket is not subject to disturbance by floods. The discussion of the impossibility of grouting sandstone to prevent filtration of water through the body of the rock is interesting. Apparently, grout can be used only to close fissures and not to tighten a naturally porous rock.

Mr. Grunsky again warns against the possibility of placing too much confidence in experimental data. He thinks that it is better to eliminate uplift than to make assumptions concerning its magnitude. Certainly no issue can be taken with this attitude, but all the available evidence seems to indicate that present good practice in the grouting and draining of foundations does not eliminate uplift.

Because so many of the discussions have warned against over-confident use of experimental data a word of counter caution may not be out of place. To discredit research work too seriously is unwise, and militates against progress. Data of the kind submitted in this paper are intended to add to the store of knowledge of past experiences, and in this way to assist in the rendering of "sound personal judgment".

With an intricate and variable problem of this kind it is too much to expect that a simple set of rules can be formulated which will enable an inexperienced man to pass on the foundation for an important dam. This is the possibility which seems to be causing anxiety, and it is right that it should be guarded against; but in doing so it should not be forgotten that depending too implicitly upon "personal judgment" has led to more than one disaster.

The most essential new points brought out in the discussions may be summarized as follows:

1.—Foundation conditions at a given site are likely to vary widely, often preventing concordance of observations. This can only be counterbalanced by a large number of observations. Readings taken at one or two points are not dependable, as they may contain local influences.

2.—When a new dam site is being investigated it is largely a matter of judgment whether or not it is similar to another site for which uplift conditions have been determined. With a greater accumulation of data engineers may be able to judge this factor with more assurance.

3.—An attempt should be made to improve present knowledge of means for controlling underflow and uplift. Grouting and drainage, as ordinarily practiced, reduce uplift, but do not eliminate it.

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Paper No. 1718

HYDRAULIC STUDIES AND OPERATING RESULTS ON THE MIAMI FLOOD CONTROL SYSTEM*

By C. H. Eiffert,† M. Am. Soc. C. E.

With Discussion by Messrs. C. S. Bennett, Morris Knowles, Walter M. Smith, B. F. Jakobsen, K. L. De Blois, E. S. Lindley, C. S. Jarvis, E. W. Lane, Charles H. Paul, and C. H. Eiffert.

Synopsis

The flood control system of the Miami Conservancy District, a description of which has been published,‡ consists of a combination of retarding basins and channel improvements. The dams and retarding basins are five in number, while channel improvements were carried out in eight cities and towns. The purpose of this paper is to state as fully as possible the results obtained thus far (1928) in the operation of the works of the District. The observations and measurements made to check the design are recorded.

The complete system has been in operation since 1922. Sufficient time therefore has not elapsed to make definite conclusions possible in all respects; but it is thought best to report at the present time and make known such information as is now available.

In general, the assumptions on which the designs of dams, conduits, and channels were based, have been confirmed. The discharge of the conduits under given heads has been somewhat larger than the quantities used in the calculations, but this was expected while the conduits were new. The outlet structures have fully met the requirements for dissipating the energy of the water as it leaves the conduits.

The improved channels at Dayton and Hamilton, Ohio, have maintained themselves better than was anticipated. The construction of cut-off channels

* Published in May, 1928, *Proceedings*.
† Chf. Engr., The Miami Conservancy Dist., Dayton, Ohio.
‡ *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 1503.

has been justified. The dams at Dayton and Hamilton together with the gravel plants have greatly reduced the cost of channel maintenance.

Valuable information has been secured from the rainfall and run-off records. Additional information could be secured by the use of a number of recording rain gauges and recording back-water gauges at the dams.

Design

The flood-control works of the Miami Conservancy District are designed for a maximum flood 40% greater than that of 1913, although the studies which were made indicated that this maximum was larger than could occur in this locality. Should such a flood occur, however, it would just reach the spillways of the dams and in the improved channels it would reach maximum heights about 3 ft. below the tops of the levees. The system is so balanced that the maximum discharge through the conduits in the dams will not be greater than can be carried safely by the improved channels below. An important feature of the design is the fact that the flow through the conduits is not controlled by gates. These conduits are smooth open tunnels, the discharge being entirely automatic.

Some of the assumptions used in the design of the conduits and of the improved channels will be restated as it was to check these that measurements, observations, and investigations have been carried on.

Conduits.—The value of the roughness factor, n, in Kutter's formula used in the design of the conduits was 0.013. It was well known, of course, that the value for smooth concrete conduits was often lower than this, even as low as 0.010. It was thought possible, however, that the concrete surfaces would deteriorate somewhat in time, therefore 0.013 was considered a proper mean between conditions in the newly built conduits and those that might exist in the future.

In calculating the discharge through the conduits it was assumed that the only loss of head necessary to be considered was that due to friction. Every effort was made to design entrance and outlet portions of the structure to eliminate losses at these points.

In making the original discharge calculations the head was calculated from the top of the outlet end of the conduits. S. M. Woodward, M. Am. Soc. C. E., has reached the conclusion* that this head should be computed from a point somewhere between the extreme top and the average top of the outlet, although the exact point cannot be determined. In recalculating the discharge curves an elevation half way between the top and the average top has been used.

River Channels.—In designing the channel improvements, values of n from 0.0225 to 0.025 were adopted after a careful study of available data. The cross-section of the improved channels, in the more extensive channel improvements at Hamilton and Dayton, is a modified trapezoid (Fig. 1). This shape concentrates the low-water flow and is also satisfactory for high water. On the Miami River channel the width, *B-E*, varies from 530 to 650

* Technical Repts., The Miami Conservancy Dist., Pt. VII, p. 144.

ft.; the width, *C-D*, is 150 ft. in nearly all cases. The depth of maximum flow between Lines *A-F* and *C-D* is from 20 to 34 ft. Points *B* and *E* are, as a rule, 8 ft. above Line *C-D*. The low bottom, Line *C-D*, varies in its location relative to the sides. Where the channel is straight, it is in the center; on curves, it is nearer the concave bank.

Stream Flow Measurements

A most important factor in the hydraulic computations are the stream flow measurements. On this work the latest improved methods, as developed by the U. S. Geological Survey, are being used throughout. Measurements are made from cableways at the dams and from bridges in the cities. Price current meters with 50-lb. weights of the new low-resistance shape are used. Meter and weight are suspended by a single $\frac{1}{8}$-in. airplane cable, which greatly reduces the resistance due to the former double insulated cable. As the weight and meter cannot be raised and lowered conveniently by hand with a $\frac{1}{8}$-in. wire, a small windlass is attached to each of the cable cars and a light, portable, folding derrick with a windlass is used on the bridges.

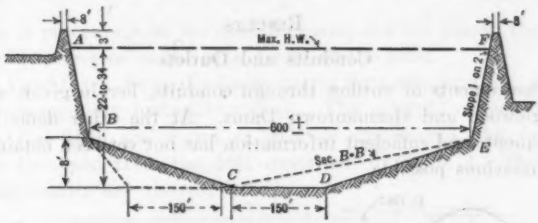

Fig. 1.—Typical Section of Improved Channel, The Miami Conservancy District.

Two men are required to take flood measurements. In most cases the use of the heavy weight and small cable eliminates the necessity of making corrections for the vertical angle; such corrections, however, are made when necessary. Current-meter readings are taken at 0.2 and 0.8 of the depth at 8 or 10-ft. intervals. Every possible precaution has been taken to eliminate error and the results appear to be very consistent.

In co-operation with the U. S. Geological Survey automatic recording gauges have been installed below the five dams and at the City of Hamilton. An automatic long-distance recorder for the river stage at Dayton is located in the District Office. Ordinary staff gauges, which are read daily, are maintained at a number of other locations by the District, the U. S. Geological Survey, and the U. S. Weather Bureau. The locations of all rain and river gauges are shown on Fig. 12.

The automatic gauges at the dams and at Hamilton are placed in concrete houses. The cost of these houses with their pipe intakes varies from $400 to $800, according to the nature of the excavation, length of intake, and height of house. The instruments themselves, known as the Au type of water-stage recorder, cost $200 each. They are very reliable, requiring attention not more than once a month. The pencil making the record is moved by

a float in the well under the gauge house and the motion of the paper on which the record is made, is controlled by a weight-driven clock.

The first two installations were made at the expense of the District. Later ones were made by the District while the instruments were furnished by the U. S. Geological Survey. All records obtained are used jointly by the District and the U. S. Geological Survey, the same being true of records obtained by the Geological Survey in other parts of the valley. Thus, more complete records are obtained than would be possible with either organization working alone.

The automatic recorder in the Dayton Office is a Stevens long-distance water-stage recorder. It consists of a sending apparatus located on a pier of the Main Street Bridge, with a recording apparatus located in the office of the District, all operated by 110-volt current. This instrument, while it is fairly satisfactory, requires more attention than the others.

Some special staff gauges have been installed in Dayton and Hamilton for the purpose of obtaining surface slopes during high water. The results will be used for making further calculations of the coefficient of roughness, n.

RESULTS

Conduits and Outlets

The measurements of outflow through conduits, herein given, are confined to the Englewood and Germantown Dams. At the other dams high stages are less frequent, and sufficient information has not yet been obtained to make definite conclusions possible.

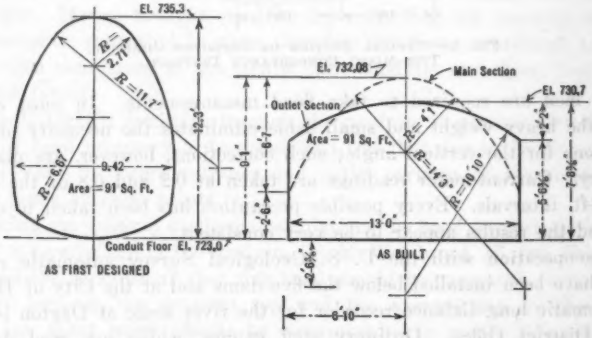

FIG. 2.—GERMANTOWN CONDUITS.

In Figs. 2 and 3 are shown comparisons of the shapes and sizes of the conduits used in the calculations for the Official Plan of the District, with those that were finally adopted and built. The areas are practically the same but, in the conduits as built at the Englewood and Germantown Dams, the sections change near the outlets (to help spread the jet) so that the top of the end of the conduit is 4.6 ft. lower at Germantown, and 4.44 ft. lower at Englewood, than in the original computations. This change, together with the fact that the point of zero head is being taken a little below the top of the

outlet portal rather than at the top, as was done originally, gives a greater head and, consequently, greater velocity and discharge for the same headwater elevations. The difference is most apparent for the lower heads. It

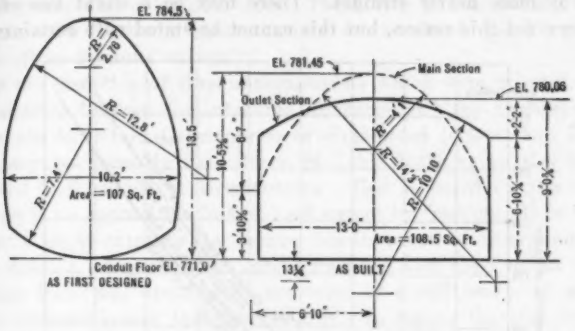

FIG. 3.—ENGLEWOOD CONDUITS.

becomes less in percentage as the head increases, for the reason that the theoretical velocity increases as the square root of the head.

In Figs. 4 and 5 are shown discharge curves for these conduits, and Table 1 gives the corresponding hydraulic values. The original curves used in the design with $n = 0.013$ are shown as well as new curves, the formulas for which were developed from the data contained in Table 2. The results of actual measurements are also platted.

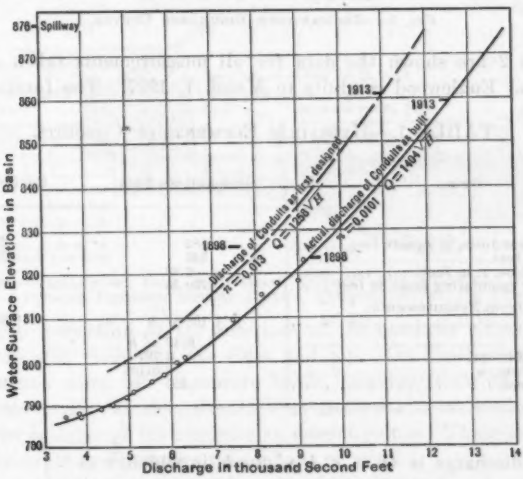

FIG. 4.—ENGLEWOOD DISCHARGE CURVES.

The roughness coefficient, n, is slightly greater for the Germantown than for the Englewood conduits (Table 1). This fact is difficult to explain,

as the interiors of the conduits at the two dams appear to be equally smooth. The entrance to the conduits at Germantown, however, is at one side of the valley and is curved; while that at Englewood is near the middle of the dam and is more nearly straight. There may be a slight loss of head at Germantown for this reason, but this cannot be stated with certainty.

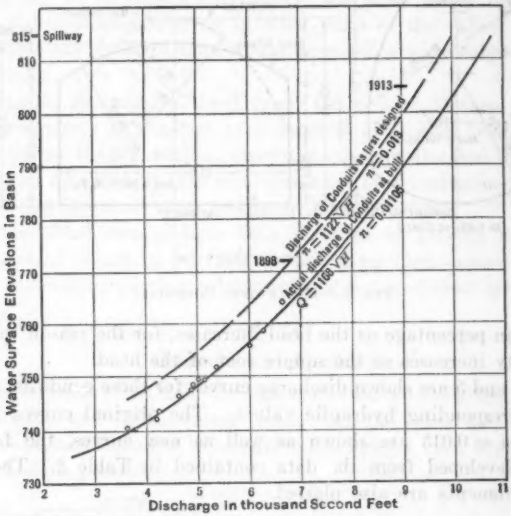

FIG. 5.—GERMANTOWN DISCHARGE CURVES.

In Table 2 are shown the data for all measurements taken at the Germantown and Englewood conduits to March 1, 1927. The formula used for

TABLE 1.—HYDRAULIC ELEMENTS OF CONDUITS.

Item.	Germantown Dam.	Englewood Dam.
Given Data:		
Area, A, two conduits, in square feet	182	217
Length, L, in feet	546	712
Hydraulic radius, r, in feet	2.43	2.71
Zero point for calculating head, in feet	730.35	779.7
Values Derived from Measurements:		
Discharge, Q	$1\ 168\sqrt{h}$	$1\ 404\sqrt{h}$
Velocity, V	$6.42\sqrt{h}$	$6.47\sqrt{h}$
Discharge coefficient, c	0.797	0.804
Kutter coefficient, n	0.01105	0.0101
Chezy coefficient, C	159	176

the conduit discharge is $Q = c\,A\sqrt{2\,g\,h}$, in which, $c = \sqrt{\dfrac{1}{1 + \dfrac{2\,g\,L}{C^2\,r}}}$; A is the cross-sectional area of the conduits, in square feet; h is the head; L, the length of the conduits, in feet; C, the Chezy coefficient; g, the acceleration

due to gravity; and r, the hydraulic radius. The values of c in Table 2 were obtained from the other factors, all of which were known. The values of n were then calculated. The final discharge formulas were developed by using the average value of c for each dam. These may be changed slightly in the future as more discharge measurements are obtained and as the interior surfaces of the conduits change.

With the exception of three measurements which were rejected, the maximum variation between the measured discharge and the discharge given by the formula for either Germantown or Englewood is less than 3 per cent.

As computed from the data shown in Table 2, the value of n is 0.0105 at Englewood and 0.01105 at Germantown. This is based on the assumption that there is no appreciable loss of head except by friction. It is quite probable that a slight entrance loss occurs, but its practical determination would be very difficult. Any such loss is now included with the friction loss and its separation therefrom would make the value of n still lower. A much lower value is improbable and, therefore, it is fair to assume that the entrance loss is insignificant.

The dissipation of the energy of the water as it leaves the conduits at a high velocity has been satisfactorily accomplished by means of the hydraulic jump and the outlet structure. For instance, at the Germantown dam, in the flood of September 2, 1926, the velocity in the conduits at the time the discharge measurement was taken, was 34.6 ft. per sec. At the end of the outlet structure the average velocity was only 7.9 ft. per sec. In Fig. 6 are shown the elevations of the tail-water for various heads at the Germantown Dam.

FIG. 6.—GERMANTOWN DAM, THE MIAMI CONSERVANCY DISTRICT, HIGH-WATER PROFILE THROUGH OUTLET WORKS, SHOWING HYDRAULIC JUMP.

A careful inspection of the interiors of the conduits shows the finishers' brush marks still visible on the sides and top. On the bottom, the surface finish has been worn off. Accurate levels, however, show that the wear is scarcely measurable at this time. Very accurate cross-sections have been taken of the interior of the conduits at several points. These will be repeated annually in order to determine the amount of wear.

Re-Routing Floods.—In making the original studies for the retarding basins a number of past floods were routed through these basins.* The

* A complete description of the routing of floods is given in "Hydraulics of the Miami Flood Control Project", by Prof. Sherman M. Woodward, Technical Repts., The Miami Conservancy Dist., Pt. VII.

maximum elevations reached and the length of time required for the basin to empty were calculated for each flood.

From the information now available it is evident that, due to the greater capacity of the conduits, these floods would not rise as high in the basins as calculated and that the time required to empty the reservoirs would be reduced. The 1913 and the 1898 floods, therefore, were re-routed through the Englewood Basin. The calculations show that the maximum elevation for the 1913 flood would be reduced 1.9 ft. and for the 1898 flood, 2.7 ft.

TABLE 2.—DISCHARGE DATA.

Date.	Head-water elevation, in feet.	Total head, h, in feet.	Discharge, in cubic feet per second.	Theoretical V, $V=\sqrt{2gh}$, in feet per second.	Actual V in conduits, in feet per second.	V at end of outlet works, in feet per second.	Value of c.	Value of n.
	GERMANTOWN DAM.*							
Jan. 11, 1924....	749.5	19.15	5 098	35.1	28.0	....	0.80	0.0109
Jan. 16, 1924....	740.15	9.80	3 470	25.4	19.1	....	‡0.75	.....
Mar. 29, 1924....	764.2	33.85	6 640	46.7	36.5	....	0.78	0.0117
June 6, 1924....	749.8	19.45	5 038	35.4	27.7	....	0.78	0.0117
June 27, 1924....	752.0	21.65	5 365	37.4	29.4	....	0.79	0.0113
Sept. 19, 1925....	743.5*	13.15	4 253	29.1	23.4	....	0.81	0.0106
Oct. 2, 1925....	748.9	18.55	4 936	34.5	27.2	....	0.79	0.0113
Nov. 13, 1925....	759.0	28.65	6 294	42.9	34.5	....	0.805	0.0107
Jan. 18, 1926....	740.25	9.90	3 747	25.3	20.6	7.0	0.81	0.0106
Feb. 25, 1926....	740.5	10.15	3 653	25.6	20.2	6.9	0.79	0.0113
Apr. 8, 1926....	746.75	16.40	4 617	32.5	25.4	....	0.78	0.0117
Sept. 2, 1926....	756.1	25.75	6 019	40.8	33.0	7.8	0.81	0.0106
Oct. 29, 1926....	742.5	12.15	4 196	28.0	23.0	6.8	0.82	0.0102
Average.....	.....	.....	.....	....	....	....	0.797	0.01105
	ENGLEWOOD DAM.†							
May 16, 1923....	791.5	11.80	4 920	27.6	22.7	....	0.82	0.0095
Dec. 14, 1923....	790.0	10.30	4 630	25.8	21.4	....	0.83	0.0092
Mar. 26, 1924....	787.3	7.60	3 880	22.2	17.9	....	0.81	0.0099
Mar. 31, 1924....	809.5	29.80	7 497	43.8	34.5	....	0.785	0.0108
Apr. 1, 1924....	802.6	22.90	6 102	38.4	28.2	....	‡0.735	.....
June 9, 1924....	817.8	38.10	8 543	49.5	39.4	....	0.80	0.0103
June 10, 1924....	822.9	43.20	9 140	52.7	42.2	....	0.80	0.0103
Apr. 23, 1925....	795.2	15.50	5 560	31.6	25.6	....	0.81	0.0099
Nov. 9, 1925....	787.5	7.80	3 829	22.4	17.6	3.4	0.785	0.0108
Nov. 13, 1925....	796.7	17.00	5 866	33.1	27.0	....	0.82	0.0095
Nov. 14, 1925....	799.8	20.15	6 155	35.9	28.3	4.3	0.79	0.0106
Nov. 16, 1925....	793.0	13.30	5 100	29.3	23.5	3.8	0.80	0.0103
Feb. 26, 1926....	788.9	9.20	4 323	24.4	19.9	3.6	0.82	0.0095
Apr. 9, 1926....	801.05	21.35	6 345	37.1	29.3	4.7	0.79	0.0106
Oct. 6, 1926....	786.9	7.20	3 501	21.5	16.1	3.4	‡0.75	.....
Oct. 30, 1926....	793.0	13.30	5 084	29.3	23.4	4.1	0.80	0.0103
Average..... ...	.....	.....	.....	....	....	....	0.804	0.0101

*Discharge formula from these data, $Q = 1\,168\sqrt{h}$.

†Discharge formula from these data, $Q = 1\,404\sqrt{h}$.

‡Rejected.

Drift.—When the plans for the dams were first developed, it was thought that some kind of drift barriers would have to be provided above the con-

duits, but, later, it was decided that these would be unnecessary. Experience thus far indicates that they are not needed. Drift reaches the conduits mostly during the early stages of a flood. During the intermediate and high stages the velocity in the basin is very low and the drift is carried about by the wind. That reaching the conduits during the first stages is usually light material and passes through easily. By the time the water is deep enough to carry large trees, the velocity has been reduced so that very few of these reach the dam.

There has never been any tendency to clog up the conduits. A number of times, as a flood went down, drift was left on the center wall. In all cases, however, this material would easily have passed through the conduits under a considerable head. As an extra precaution against the accumulation of drift, the caretakers go over their respective basins during the fall and winter and cut all "down timber" into short lengths that will pass through without difficulty.

Channel Improvements

The most extensive channel improvements were made at Dayton and Hamilton. The waterways were straightened, widened, and deepened and made more uniform in cross-section and regular in alignment. The condition of the channels at present is excellent, although there have been some slight changes. The stability of these channels has been greatly increased by the low concrete dams at Island Park and Wolf Creek, in Dayton, and at Twomile Creek, in Hamilton; and by the operation of gravel plants in both cities. These plants, above the limits of the improvements, dig large quantities of gravel from the river bed. The excavations thus made serve to catch most of the gravel carried down stream during floods and prevent its deposit in the improved channel. Besides saving maintenance expense, the District obtains some revenue from the sale of the gravel.

Some of the changes in cross-section of the improved channels are illustrated in Figs. 7 and 8. In Dayton there has been considerable deposit above the Island Park Dam. Fig. 7 shows a section 100 ft. above the dam (taken in 1916 before the dam was built), another in 1923, and a third in 1926. The progressive filling indicates that it will not be long until material will begin to pass over this dam and enter the channel below. However, the material deposited here seems to be rather fine and possibly may be carried on through the improved channel. Small bars have formed at other points in Dayton but, taken as a whole, the sections show the remainder of the channel to be practically as built.

At Hamilton a bar has formed below the dam, probably due partly to the gravel carried over. This material is now being removed by the Hamilton Gravel Company as part of its regular operations. Below the railroad bridge the channel widens causing a considerable deposit along the west side (Fig. 8). The alignment here was made to fit the original bank for economy in construction. Such a deposit would not occur in a channel of uniform width and more regular alignment. A flood somewhat larger than those which have occurred, may remove it.

A flexible revetment is used as protection against erosion. This consists of concrete blocks, 5 by 12 by 24 in., each reinforced and cast with two transverse holes. The blocks were manufactured at a central plant. They are held together by ½-in. galvanized steel cables, the ends of which are embedded in the toe-wall of the monolithic slope revetment on the land side, and in the larger blocks, cast in place, on the river side. A detail drawing of this revetment has been published.* As scouring occurs, this revetment settles to a depth where further scour is prevented.

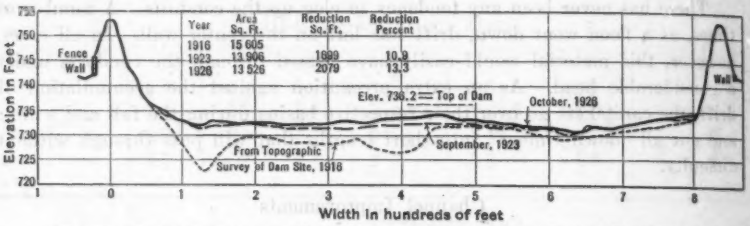

FIG. 7.—CROSS-SECTIONS OF MIAMI RIVER 100 FEET ABOVE ISLAND PARK DAM.

Value of n for Channels.—The discharge measurements made thus far in Dayton and Hamilton show the coefficient of friction, n, to vary from approximately 0.015 to 0.0285. The larger value was obtained at Dayton with a gauge height of 9 ft. At this stage the water just covered the entire bottom from toe to toe of the levees, the depth varying from 0 at the sides to 8 or 10 ft. in the center, over a width of about 600 ft. The lowest value of n obtained was 0.015; this was above the Main Street Bridge, at Hamilton, at a 12.4-ft. stage. The channel here is straight, has a uniform width of 520 ft., and a smooth bottom practically free from grass or weeds, with concrete revetment on the slopes.

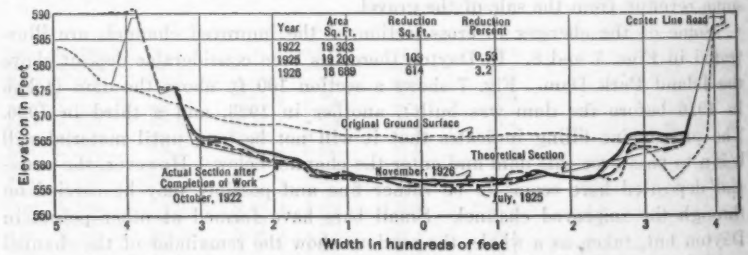

FIG. 8.—CROSS-SECTIONS OF MIAMI RIVER AT STATION 72 + 00, HAMILTON, OHIO.

The detailed procedure in making two determinations of the value of n was as follows: In the storm of March 19 to 21, 1927, the stage of the river at Dayton stayed close to the 12-ft. mark for about two days. The discharge was measured at the Main Street Bridge while the average gauge height was 12 ft. This discharge, 38 300 sec-ft., was used in the computations. Permanent gauges reading to sea-level elevations had previously been installed above and below the Main Street Bridge as shown in Fig. 9(*a*). These gauges were

* *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), Fig. 35, p. 1508.

read before, during, and after the discharge measurement, the entire set being read five times; three times by the man who took the discharge measurements, and once each by two other men independently.

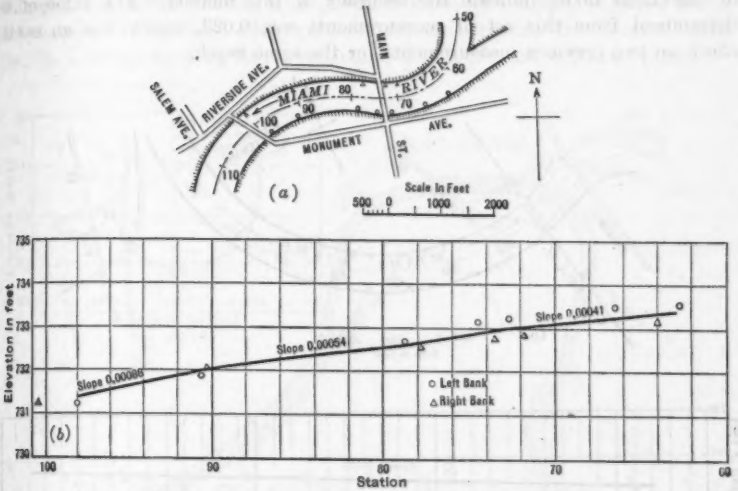

FIG. 9.—HIGH-WATER PROFILE, STATION 63 + 00 TO STATION 98 + 00, THE MIAMI CONSERVANCY DISTRICT, MARCH 21, 1927.

It was recognized that the chief difficulty lay in obtaining accurate slope measurements, and considerable care was taken in obtaining the readings. As each set of slope gauge readings was made, the Main Street river gauge was also read. As there was only a small variation in the gauge height during this time, the surface slopes were practically parallel. Therefore, all slope gauge readings were corrected by addition or subtraction to reduce them to a 12-ft. stage. The five readings for each gauge were then averaged to obtain the final figure. The slope profile was then platted for both sides of the channel, and the slope used in the determination of n was taken as a mean between the surface elevations on the two sides.

This surface slope was used in determining the cross-sectional areas. Accurate cross-sections at 100-ft. intervals were available. Only part of these were used, however, as the areas are quite uniform and the changes gradual. Fig. 9(*b*) shows the final water surface profile. To determine n, three reaches were used, as shown. The slope, hydraulic radius, and velocity having been determined, the value of n was taken from a diagram for the solution of Kutter's formula. For this part of the river it averaged 0.027.

Perhaps the most satisfactory determination was made on a reach of about 3 000 ft. below Stewart Street, at Dayton. Here, the channel is of uniform section throughout and of uniform curvature, with a radius of 4 093 ft. on the center line. The profile in this case was determined by driving stakes flush with the water surface every 200 or 300 ft. on both sides of the channel. While these stakes were being driven the gauge height at Main Street was

11.9 ft. Levels were read on the stakes and the elevations corrected for the 12-ft. stage at which the discharge measurement was taken. The resulting profile is shown in Fig. 10(*b*). The uniformity of the slope and the number of elevations taken indicate the accuracy of this method. The value of n determined from this set of measurements was 0.023, which was an exact check on two previous measurements for the same reach.

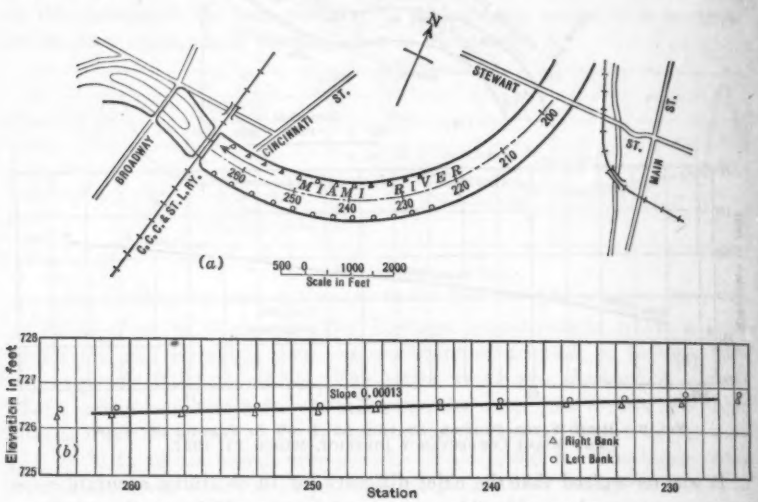

FIG. 10.—HIGH-WATER PROFILE, DAYTON, OHIO, MARCH 21, 1927.

Taking the channels at Dayton and Hamilton as a whole, the average value of n appears to be below that used in the design, and therefore the channel capacity will be somewhat greater than provided by the plan. It is reasonable to assume that at higher stages the values of n will be less than those obtained from the measurements made thus far.

Cut-Off Channels.—At a number of points where more extensive channel improvements were not warranted, cut-off channels were required to facilitate the flow. These were built of small cross-section on the assumption that they would be enlarged by scouring. Each of them reduces the length of the channel, thereby increasing the grade and the velocity. Such channels are located at Troy, Dayton, Poasttown, Middletown, and Hamilton.

The Troy cut-off channel (Fig. 11) is 4 000 ft. long. The original bottom width was 40 ft. which has increased to about 65 ft. Depth and alignment have been well maintained. It assists greatly in passing the water through the City of Troy and its capacity, of course, will continue to increase. The protection levee for the city is located close to this channel for some distance and as the channel is widening about equally on both sides some bank protection may be necessary along the levee.

In constructing this channel the bank nearest the levee was cut on a 2 to 1 slope while the other bank was made vertical. It was thought that cutting

would occur mostly on the vertical side and that the 2 to 1 slope would remain intact. The cutting has actually been about equal on the two sides.

The cut-off channel at the Broadway Bridge, just below Dayton, has not enlarged, due to the fact that it is rather close, and nearly parallel, to the old channel. It has been effective, however, in increasing the channel capacity. In a severe flood no doubt considerable enlargement will take place.

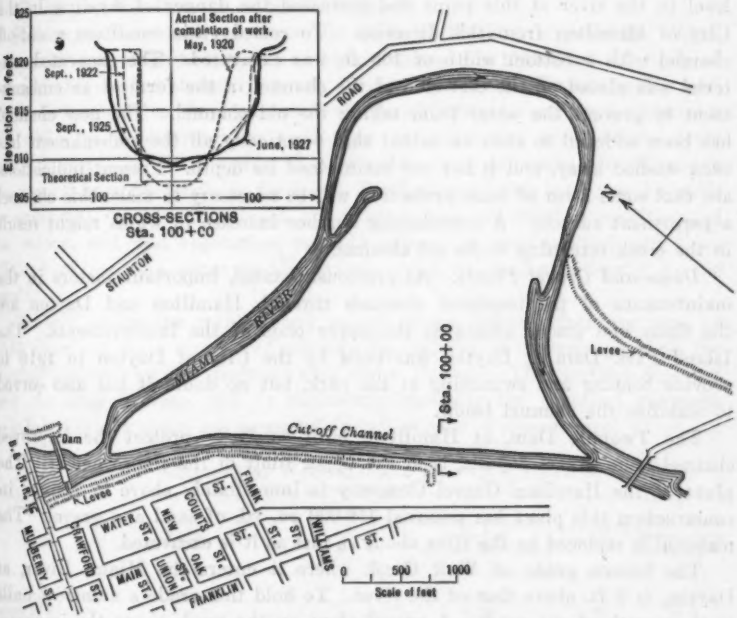

FIG. 11.—CUT-OFF CHANNEL, TROY, OHIO.

A rather remarkable result was obtained north of Middletown on a bend in the river just above the Poasttown Bridge. Here, no excavating was done. The timber was cleared from a strip where a cut-off channel would naturally be located and the river has cleared the remainder. This channel gives the river a more direct approach to the bridge and protects the road on the east, which otherwise would be endangered by scouring.

Opposite Middletown, the Miami River is in the form of a big bend. The overflow area is large, extending, in high water, to the railroad on the west. A concrete bridge about 1 800 ft. long crosses almost the entire bottom. The alignment of the old channel greatly retarded the flow and considerable scour was taking place along the east bank. A cut-off channel with a bottom width of 40 ft. was constructed. At first, only a small part of the flow passed through this channel, but it has been enlarged by floods to such an extent that the cross-sectional area has increased 110% and it now carries the entire low-

water flow and a large proportion of the flood flow. It has also eliminated the scouring along the east bank of the old channel. In this case the results have probably exceeded the expectations of the designers.

Prior to the execution of the District's plan a bad situation existed north of Hamilton, near the mouth of Fourmile Creek, which has a drainage area of 178 sq. miles. The direction of flow of this creek was almost directly opposite that of the river. The retarding action resulting raised the water level in the river at this point and increased the danger of flooding in the City of Hamilton from this direction. To remedy this condition a cut-off channel with a bottom width of 100 ft. was excavated. The excavated material was placed on the east side of the channel in the form of an embankment to prevent the water from taking the old channel. The new channel has been widened to such an extent that practically all the embankment has been washed away, and it has not maintained its depth. Present indications are that some form of bank protection will be necessary to make this channel a permanent success. A considerable further increase in width might result in the creek returning to its old channel.

Dams and Gravel Plants.—As previously stated, important factors in the maintenance of the improved channels through Hamilton and Dayton are the dams and gravel plants at the upper ends of the improvements. The Island Park Dam at Dayton was built by the City of Dayton in 1916 to provide boating and swimming at the park, but no doubt it has also served to stabilize the channel below.

The Twomile Dam, at Hamilton, built solely to protect the improved channel from gravel deposit, is at the upper limit of the improvement. The plant of the Hamilton Gravel Company is immediately above it. Since its construction this plant has removed 156 000 cu. yd. of sand and gravel. The material is replaced by the river about as fast as it is excavated.

The bottom grade of Wolf Creek where it enters the Miami River, at Dayton, is 5 ft. above that of the river. To hold this grade a dam was built at the mouth of the creek. A gravel plant on the creek above the improvement helped to maintain the channel. This plant was abandoned after about 42 000 cu. yd. had been removed. Gravel was not carried down fast enough to supply the plant, for the reason that material was being taken out in large quantities about ½ mile farther up stream.

Gravel plants were located also on Mad River, below Webster Street and below Findlay Street in Dayton. The former has been abandoned, at least temporarily, because the supply was exhausted. It removed a total of about 57 000 cu. yd. Two plants are operating below Findlay Street. They have moved approximately 222 000 cu. yd. to date (1928).

The plants operating on District property have removed from the river a total of 477 000 cu. yd. since the early part of 1922. There can be no doubt that much of this gravel would have passed down stream had it not been removed. A large part of it would certainly have stopped in the improved channels and its removal would cost probably not less than 75 cents per cu. yd.

Rainfall and Run-Off

The District maintains rain gauges at all the dams and at a number of other points. The U. S. Weather Bureau also has gauges in various cities in the Miami Valley. The reports from all these stations are used co-operatively by the District and the Weather Bureau. All rainfalls of more than 1 in. in 24 hours are reported by telephone or telegraph. Other rainfall reports are sent in weekly or monthly by mail. The map, Fig. 12, shows the location of all the gauging stations in the water-shed.

No storms have occurred, since the works of the District were completed, approaching in magnitude the one which produced the 1913 flood. A study of some of the smaller storms, however, is of value. The difference in the percentage of run-off in these storms is of special interest, because it illustrates the difficulty encountered in predicting or estimating river stages from rainfall data. In other words, it shows that it is impossible to determine the run-off by a fixed rule or formula. Predictions of this kind may be made by using the records of the past, but in each case the existing conditions of the season, soil, and vegetation, must be taken into consideration and a coefficient of common sense and judgment must be applied liberally to the result.

Table 3 shows the rainfall and run-off above Dayton, from an area of 2 525 sq. miles, for a number of storms. The rainfall is the average of all the rainfall stations on the drainage area above Dayton. This method does not give the true average, but is sufficiently accurate for practical purposes. Rainfall contour maps are prepared for all storms of importance.

TABLE 3.—Relation of Rainfall and Run-Off, Dayton, Ohio.

Date.	Average rainfall, in inches, above Dayton.	Run-off, in inches.	Run-off, in percentage of rainfall.
March 23-27, 1913	9.60	8.76	91.0
March 28-29, 1924	2.06	1.16	56.0
June 8, 1924	2.22	1.73	78.0
Sept. 12-13, 1925	3.20	0.04	1.25
Nov. 12, 1925	1.43	0.66	46.0
Nov. 26-27, 1925	1.29	0.20	15.6
April 7-8, 1926	1.23	0.84	68.0
March 19-21, 1927	3.44	2.31	67.0

The run-off is determined by reducing the daily gauge heights at Dayton to discharges, in cubic feet per second, and platting a curve of the daily discharges for the storm period. The area included between this curve and a line showing the normal discharge is then measured and the result reduced to inches of rainfall over the drainage area.

A study of the records of the storms of September 12 and 13, 1925, and March 19 to 21, 1927, discloses a remarkable contrast. The average rainfall above Dayton for the two storms was 3.20 and 3.44 in., respectively. The run-off for the storm of September 12 and 13, 1925, was 0.04 in., or 1.25% of the rainfall. For the storm of March 19 to 21, 1927, the run-off was 2.31 in., or 67% of the rainfall.

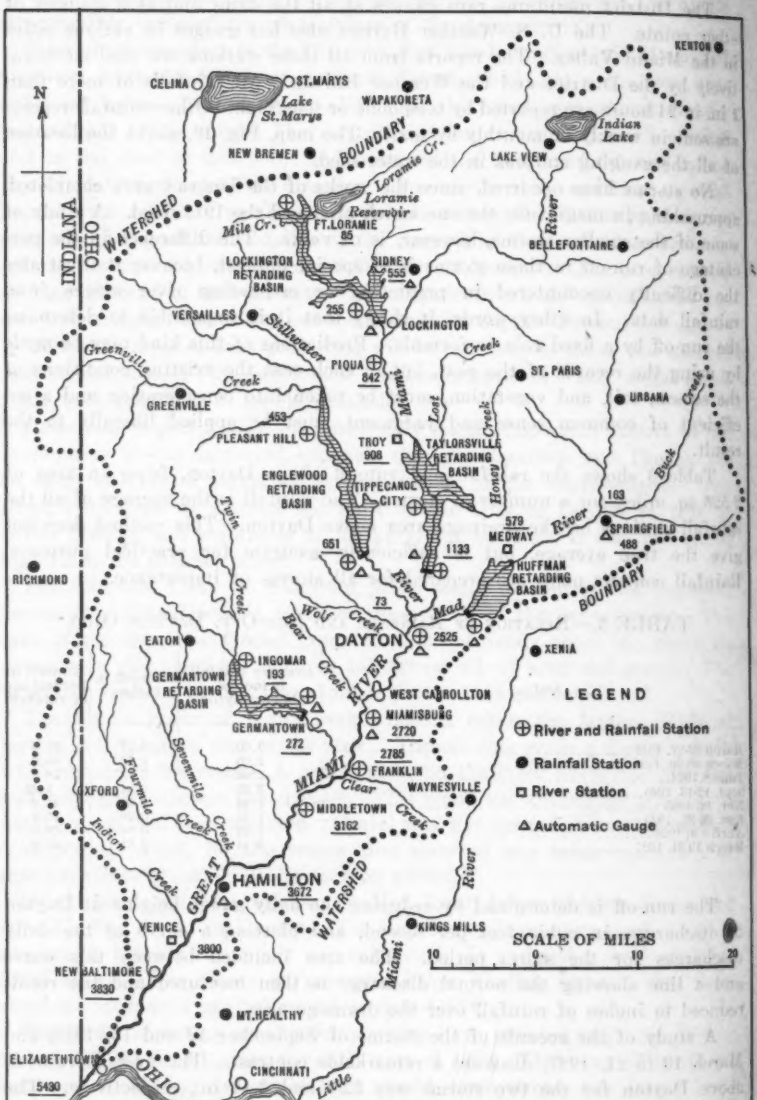

FIG. 12.—DRAINAGE AREA, SHOWING GAUGING STATIONS, MIAMI RIVER, OHIO.

The storm of September 12 and 13, 1925, was of shorter duration. The precipitation for 24 hours at several stations was greater than for any 24-hour period of the 1913 flood. Other things being equal, this would have caused the run-off for this storm to be the greater of the two. However, there was no back-water at any of the dams and the maximum stage reached at Dayton was 2.9 ft.

After the storm of March 19 to 21, 1927, all the dams had back-water, the depths being, at Englewood 48 ft.; Taylorsville, 26 ft.; Huffman, 16 ft.; Lockington, 32 ft.; and Germantown, 29 ft. The maximum stage recorded at Dayton was 12.8 ft. and at Hamilton, 14.3 ft.

The rainfall and run-off curves for both storms are shown in Fig. 13. Unquestionably the difference in run-off was due to the difference in soil conditions. The storm of September, 1925, came at the end of the growing season and following a dry period during which the transpiration and evaporation had been abnormally great. At the beginning of this period there was a deficiency in rainfall of about 5 in. These factors combined to give the soil a tremendous storage capacity. Preceding the storm of March, 1927, there had been little evaporation and there was no growing vegetation to consume moisture. To November 1, 1926, there had been an excess of 5.79 in. in rainfall since the beginning of the year.

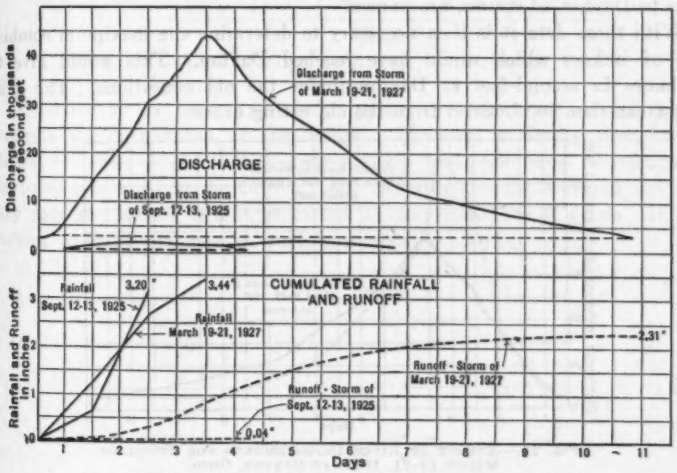

FIG. 13.—STUDY OF RAINFALL AND RUN-OFF, DAYTON, OHIO.

On October 2, 1925, an unusually heavy rain occurred over the drainage area of Clear Creek. This creek, which enters the Miami River just below Franklin, has a drainage area of 52 sq. miles. Following the storm numerous inquiries were made from residents and the information obtained showed the average rainfall for the area to be 4.3 in., with a maximum of 5¼ in. The rainfall occurred between midnight and 9:00 A. M. Surface slopes were determined from high-water marks west of the Dixie Highway Bridge, at Franklin,

and the maximum discharge was calculated to be 10 000 sec-ft., or about 200 sec-ft. per sq. mile. It was not possible to obtain sufficient data to determine the total amount or percentage of run-off.

After high water under present conditions the question is often asked: "What would have happened under the old conditions?" The answer involves an interesting study. It is assumed that the maximum stage at Dayton would be produced by the maximum combination of the inflows into the Englewood, Taylorsville, and Huffman Basins. The run-off from the area below these basins, which would affect the maximum stage at Dayton, will, in most cases, be negligible. In practically all storms this run-off passes Dayton before the maximum stage is reached. It is, therefore, disregarded in these calculations.

During high water the back-water gauges at the dams are read every 2 hours. From the contour maps of the basins a tabulation of the storage capacities of each basin, for various elevations, has been made. From these and the 2-hour readings, together with the outflow record, the total amount of inflow for each 2-hour period can be determined. From this the maximum inflow, in second-feet, is obtained.

From former studies the rate of flow in these rivers was found to be from 3½ to 4 miles per hour. The distances along the rivers from the three dams to Dayton, of course, are known.

With these data it is then necessary to determine the maximum combination of inflows which would have reached Dayton. This would give the discharge in second-feet at Dayton under the old conditions. The gauge height can then be obtained from the old rating curve.

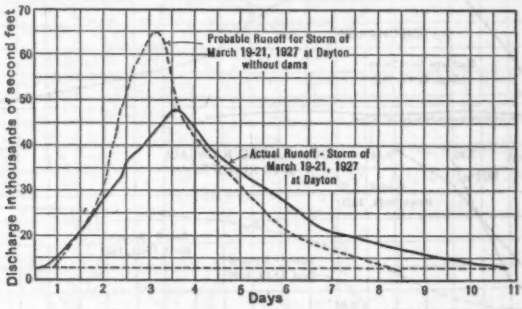

FIG. 14.—EFFECT OF RIVER IMPROVEMENTS FOR STORM OF MARCH 19-21, 1927, AT DAYTON, OHIO.

This method was applied to the storm of March 19 to 21, 1927, the results being shown in Fig. 14. The solid line shows the run-off as it actually occurred at Dayton; the dotted line represents the run-off as it would have taken place before the flood-control works were built. It is to be noted that the peak discharge under former conditions would have been 45% greater and would have occurred about 5 hours earlier.

The maximum gauge height at Dayton was 12.8 ft. In the old channel the actual elevation of the water surface would have been 6.9 ft. higher

The difference in the two peaks at Dayton will be much more pronounced in a larger flood for the reason that the velocities and discharges through the conduits at the dams vary approximately as the square roots of the heads.

Fig. 15 shows the inflow, outflow, and elevation curves for the March, 1927, storm at Englewood. These curves show that at this point alone the peak discharge was reduced from 22 300 to 9 000 sec-ft.

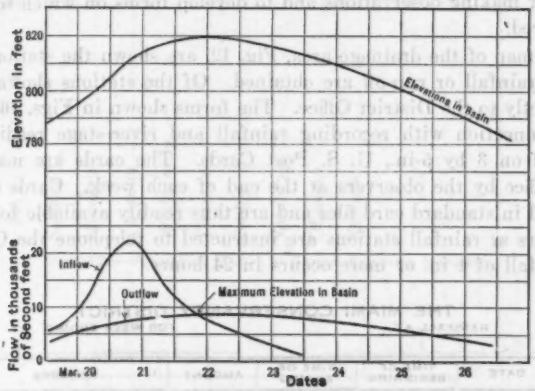

FIG. 15.—FLOOD OF MARCH 19-21, 1927, AT ENGLEWOOD, OHIO.

As stated at the beginning of this paper, the experience gained thus far confirms the correctness of the design. However, sufficient time has not elapsed to make definite conclusions possible along all lines. Investigations are to be continued in the future. The opportunities for securing the necessary data are limited to a large extent by the occurrence of floods. It is the purpose to make the most of such opportunities as they occur.

DISCUSSION

C. S. BENNETT,* M. AM. SOC. C. E. (by letter).—Some details of the manner of obtaining and recording data for use in studies of the operation of the flood control system of the Miami Conservancy District may be of interest. In order to systematize the work it became necessary to adopt certain standard methods for making observations and to develop forms on which to record the data gathered.

On the map of the drainage area, Fig. 12, are shown the stations at which records of rainfall or run-off are obtained. Of the stations shown, seventeen report directly to the District Office. The forms shown in Figs. 16 and 17 are used in connection with recording rainfall and river-stage readings. They are printed on 3 by 5-in., U. S. Post Cards. The cards are mailed to the District Office by the observers at the end of each week. Cards of this size may be filed in standard card files and are thus readily available for reference. All observers at rainfall stations are instructed to telephone the Office whenever a rainfall of 1 in. or more occurs in 24 hours.

THE MIAMI CONSERVANCY DISTRICT

RAINFALL AT _____ FOR WEEK ENDING _____ 19__

DATE	TIME OF BEGINNING	TIME OF ENDING	AMOUNT	REMARKS

OBSERVER_____

FIG. 16.—FORM FOR WEEKLY PRECIPITATION REPORTS.

Studies are made of every major storm that occurs. To facilitate the collection and use of data for these studies forms such as Figs. 18 and 19 were devised. Fig. 18 is used by the observers at the dams. From the time that the outlet conduits are half full until the stage falls again to that elevation, readings are made every 2 hours at the gauges above and below the dam. A series of staff gauges have been placed on or near the up-stream slope of each dam at convenient locations, the range of these gauges extending from the elevation of the mid-point in the conduit to spillway crest elevation. Below the dams are automatic recording gauges, near which staff gauges are located in the stream. The latter are read by the observers during times of high water, such readings serving as a check on the automatic recorder. From these it is possible to construct inflow and outflow curves for the storms to be studied.

* Engr., The Miami Conservancy Dist., Dayton, Ohio.

Form M-38

THE MIAMI CONSERVANCY DISTRICT

MIAMI RIVER Gaging Station, Hamilton, Ohio

Daily Readings for Week Ending *192*

DAY	GAGE	TIME
Sunday		
Monday		
Tuesday		
Wednesday		
Thursday		
Friday		
Saturday		
Observer		
Remarks:		

FIG. 17.—FORM FOR WEEKLY GAUGE READING REPORTS.

GAGE RECORD FOR _____ DAM.

Date			Date			Date		
TIME	UPPER GAGE	LOWER GAGE	TIME	UPPER GAGE	LOWER GAGE	TIME	UPPER GAGE	LOWER GAGE

Date			Date			Date		
TIME	UPPER GAGE	LOWER GAGE	TIME	UPPER GAGE	LOWER GAGE	TIME	UPPER GAGE	LOWER GAGE

At what time were conduits just full _____

What was highest point reached on upper gage _____

When _____

What was total rainfall for this storm _____

When did it start _____ When did it stop _____

FIG. 18.—FORM FOR GAUGE READING RECORDS.

Storm of_____

	ALL STATIONS					RIVER STATIONS ONLY				DAMS ONLY								
LOCATION	Drainage Area, Sq. Miles	Rainfall Average over Basin	Run-off in Inches	Max. Rainfall 24 Hrs.	Max. Run-off 24 Hrs.	Max. Gage Height	Time	Max. Discharge Second Feet	Max. Discharge Sec. Feet per Sq. Mi.	Maximum Inflow Sec. Ft.	Time	Max. Outflow Sec. Ft.	Time	Max. Elev. W. S. above Dam	Time	Max. Elev. W. S. below Dam	Time	Max. Volume in Conduits	REMARKS
Germantown	272																		
Englewood	651																		
Lockington	255																		
Taylorsville	1133																		
Huffman	671																		
Sidney	555																		
Piqua	842																		
Troy	908																		
Tipp. City	1016																		
Springfield	488																		
Dayton	2526																		
Miamisburg	2720																		
Franklin	2785																		
Middletown	3162																		
Hamilton	3672																		

FIG. 19.—FORM FOR RECORDING STORM DATA.

On the form, Fig. 19, are recorded the essential data for each storm. In connection with this a rainfall map is prepared. The observed rainfall at each station is noted on the map and rainfall contours or isohyetals are drawn in. The average rainfall and run-off are computed usually only for the Dayton Station. Average rainfall may be obtained by planimeter from the contour map or by using the mean of the recorded values above Dayton.

Data on Silt Deposits

Location_____ Date of Storm_____

Max. El. of Backwater_____ Time of Max. Stage_____

Condition of Ground

Very Dry— Dry — Wet— Very Wet—Frozen

Box No.	Elev.	Duration of Overflow	Depth in Box	Depth on Board

FIG. 20.—FORM FOR RECORDING DATA ON SILT DEPOSITS.

It has been found that the latter method is fairly accurate. The run-off is obtained by measuring, with planimeter, the area under a curve constructed from the river-stage record at Dayton for the period of the storm. Maximum run-off in 24 hours may be obtained also from this curve. Maximum discharges at the various river stations are obtained from the rating curves for those stations. Maximum inflow and outflow at the dams are obtained from curves prepared from data contained on the observers' record (Fig. 18) and from the rating curves at stations just below the dam.

A number of silt-catching boxes have been installed in two of the basins near the dams.* Soon after the subsidence of stored water after each major storm, measurements are made of the silt content within the boxes and on the projecting bases. The data thus obtained are recorded on the form similar to Fig. 20 and these are filed for future use. A number of cross-sections have been taken across the valleys near the silt boxes. At intervals of a year or more, these sections will be repeated and platted to a fairly large scale so that the quantity of silt remaining as a permanent deposit may be measured.

Discharge measurement notes, rating tables, records of daily discharge, and other data secured in connection with stream-flow records are arranged on standard forms of the U. S. Geological Survey, because such information is used jointly by the Geological Survey and the District.

MORRIS KNOWLES,† M. AM. SOC. C. E. (by letter).—This paper is an interesting contribution to the already voluminous literature on flood protection works. Its value, however, lies in the fact that it presents actual data on results of works already constructed. There has been a paucity of these data and they are the kind of which the profession should have more, as a guide to the design of future plans of controlling devastating floods.

As the author states, the period of six years since the completion of the Miami System is not long enough to warrant final conclusion, but the observations so far obtained are of value in two respects: They demonstrate that the basic formulas involved in the original studies and the method of attacking the problem have been sound. Of additional interest are the observations which have been made with great care to determine values of the roughness factor, n. Even with the data already at hand, additional values for this factor are always desirable, particularly for improved channels, as in this case.

It is to be noted that observed values of n for the improved channels ranged from 0.015 to 0.0285, but most of them were of the higher values. The low value of 0.015 has been readily explained. The originally adopted values were from 0.0225 to 0.025. The higher figures obtained from observation more nearly approach the value of 0.030 which has often been used as a general criterion for natural river channels of large sizes. Studies recently made along the Ohio River show values for this coefficient ranging from 0.035 to 0.047 in the main channel. These are, of course, in an unimproved reach of the Ohio.

* See discussion by C. H. Elfert, M. Am. Soc. C. E., on the paper entitled "Silting of the Lake at Austin, Texas," by T. U. Taylor, M. Am. Soc. C. E., in this volume of *Transactions*.

† Pres. and Chf. Engr., Morris Knowles, Inc., Pittsburgh, Pa.

The retarding basins, which formed the major parts of the Miami project, have demonstrated their effectiveness. While they are a departure from the generally accepted idea of storage basins, they have shown their value in cutting down peak flows, in distributing the flow over a longer period, and in demonstrating the fact that their operation is entirely automatic. The design of the conduits through the dams has been found to be sound and here, again, some interesting observations upon the value of n have been obtained, which check very closely those used in the design.

The engineers of the District are to be commended for thoroughness in obtaining rainfall and run-off data, and for the provisions being made for securing more of this information by additional stations. It is wisely pointed out that it is impossible to determine run-off from rainfall by any fixed rule or formula. The careful observations of all the factors entering into the determination of run-off will be of great value to those who do not have the facilities, either in time or money, to obtain the detailed field data for similar studies in other localities.

One of the most interesting features of the entire paper is that part which compares the actual run-off at Dayton, which occurred in March, 1927, with the probable run-off without the improvements. A reduction of 27% in the peak flow, which might have taken place, represents considerable less possible flood damage. The retarding basins and channel improvements along the Miami River have already proved beneficial to the District and from observed data it is apparent that the 1913 disaster can never be repeated.

Walter M. Smith,* M. Am. Soc. C. E. (by letter).—The writer has read this paper with a great deal of interest, because the dams, conduits, and spillways on that work were all designed under his direction. It is very gratifying to know that the six years of operation have justified the design and construction of the project as a whole as well as the individual units.

The writer has always had the idea that the value, $n = 0.013$, for the conduits would prove too high and that, therefore, more water would be discharged and less held in storage than the computed quantity. The concrete of the conduits was made very rich in cement—about 2 bbl. per cu. yd. of concrete—in order to secure a very dense and hard concrete that would resist abrasion and a growth of any kind of moss or other material. The material carried through the conduits during floods will naturally scour the floor somewhat, but should not affect the other parts of the surface. Therefore, the roughness factor, n, should not change very much over a long period. It is customary, in water supply or sewer computations, to assign to n a value a little higher than is ever expected in order to provide a factor of safety. In the case of the Miami Conservancy work, however, assigning too large a value to n does not give a factor of safety, but just the reverse. For example, the actual flow through the Englewood conduit, as given in the paper, is 14½% greater than as computed. Should this same ratio hold good at the Taylorsville and Huffman Dams, the total flow for the maximum flood through Dayton would be about 114 000 sec-ft. plus the accumulated flow below the dams, instead of

* Asst. Chf. Designing Engr., Div. of Waterways, State of Illinois, Chicago, Ill.

about 100 000 sec-ft. plus this flow. As the channels through the towns were designed and built to accommodate the computed flow, this additional flow might cause damage.

It is regrettable that the author does not give results of observations at the Taylorsville and Huffman sites if they have been made, because the flow from these two sites combined is 7½ times as great as that at Englewood. If none has been made, it would seem that in view of results at Englewood, observations at these two sites should be taken at as early a date as possible.

With regard to entry loss of head the conduits in all cases were built with a carefully designed bellmouth to prevent, not only loss of head at entrance, but to prevent vibration and booming caused by entrapping air just inside the entrance as would happen if the entrance had been built with a sharp edge. The vertical section through the Germantown conduit (see Fig. 6) shows this entrance with a very slight rounding, much less than it really has. As the author has not mentioned any trouble from vibration or booming at the entrances it is assumed by the writer that the bellmouth entrance is properly designed. This was one of the most serious features of the design and one of those most carefully studied.

B. F. JAKOBSEN,* M. AM. SOC. C. E. (by letter).—Table 1 shows that the roughness coefficient, n, in Kutter's formula is about 10% greater for the Germantown conduit than for the Englewood conduit and that the Chezy coefficient, C, is about 10% greater for the Englewood conduit. Kutter's formula has been derived largely from tests on open channels and the Chezy formula has been superseded by others which do not show a linear relation between r, the hydraulic radius, and h_f, the friction head.

The carefully made measurements afford an opportunity to check the formula for friction head, for water flowing with high velocity in a large conduit. On the assumption, made by the author, that the entrance loss is negligible, the total head is consumed, partly as velocity head and partly as friction, or,

$$h = \frac{v_a^2}{2g} + h_f \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

in which,

h = total head, in feet;
v_a = actual velocity, in feet per second;
$2g = 64.36$; $\sqrt{2g} = 8.023$; and
h_f = friction head, in feet.

The theoretical velocity head is $\frac{v_a^2}{2g}$ on the assumption that v_a is constant for the entire section. The friction head is,

$$h_f = \frac{m}{2g} \frac{v_a^n}{(4r)^x} L \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

in which, L = length of conduit, in feet; r = hydraulic radius, in feet; and m, n, and x are constants. For new cast-iron pipe, Unwin gives,† $m = 0.0215$;

* Cons. Engr. (La Rue & Jakobsen), Los Angeles, Calif.
† "A Treatise on Hydraulics," 1912, p. 217.

$x = 1.168$; and $n = 1.95$. Assuming, however, that $n = 2$ (since that checks closely with the results found) and combining Equations (1) and (2):

$$h = \frac{v_a^2}{2g}\left(1 + \frac{m\,L}{(4\,r)^x}\right) \quad \ldots\ldots (3)$$

Let,

$$k = \frac{m\,L}{(4\,r)^x} \quad \ldots\ldots (4)$$

so that,

$$h = \frac{v_a^2}{2g}(1 + k) \quad \ldots\ldots (5)$$

or,

$$v_a = c\sqrt{2\,g\,h} \quad \ldots\ldots (6)$$

in which, c is the author's constant in Table 1. From Equations (5) and (6) it follows that,

$$c = \frac{1}{\sqrt{1+k}}, \text{ or } k = \frac{1}{c^2} - 1 \quad \ldots\ldots (7)$$

If Equation (6) is written,

$$c = \frac{v_a}{\sqrt{2\,g\,h}}$$

and it is assumed that every value of c thus determined from corresponding measurements of v_a and h_1 is equally probable, then the principle of least squares demands, that,

$$\sum\left(\frac{v_a}{\sqrt{2\,g\,h}} - c_m\right)^2 = \text{minimum} \quad \ldots\ldots (8)$$

in which, c_m is the most probable value of the constant, c. If N is the number of tests, then,

$$c_m = \frac{1}{N}\sum\frac{v_a}{\sqrt{2\,g\,h}} = \frac{1}{N}\sum c \quad \ldots\ldots (9)$$

or c_m is the arithmetic mean of all values of c. It would probably be more correct to derive c_m from Equation (5). This requires more work and the value thus found differs only by about 0.1% from the arithmetic mean used by the author. The probable error of c_m is given by,

$$e = \frac{2}{3}\sqrt{\frac{\Sigma\,(c - c_m)^2}{N\,(N-1)}} \quad \ldots\ldots (10)$$

and the probable error of one measurement is,

$$e_1 = \frac{2}{3}\sqrt{\frac{\Sigma\,(c - c_m)^2}{(N-1)}} \quad \ldots\ldots (11)$$

The values of L and r for the two conduits are given in Table 1. From these are obtained for the Germantown conduit, $c_m = 0.79708$; $e = 0.0027$; and $k = 0.5740$, whereas for the Englewood conduit, the values are $c_m = 0.8043$; $e = 0.0026$; and $k = 0.5458$.

For the Germantown conduit the probable error of the mean is $\frac{0.27}{0.797}$ = 0.34% and if the average velocity measured is about 7 ft. (since, presumably, the high velocities in the conduits were not actually measured), the error of the average is $7 \times 0.0034 = 0.0238$ ft. per sec. and the probable error of a single measurement is $\sqrt{12}$ times that, or 0.0825 ft. per sec. For the Englewood conduit about the same results are obtained. This indicates that the measurements were carefully made and that the friction head varies nearly as the square of the velocity.

The constant, m, can now be determined from Equation (4). That gives for the Germantown conduit,

$$m = 0.5740 \times \frac{14.24}{546} = 0.01497$$

and, for the Englewood conduit,

$$m = 0.5458 \times \frac{16.14}{712} = 0.01222$$

Both values are much smaller than the value given by Unwin for cast-iron pipe, and the value for the Germantown conduit is about 20% greater than that for the Englewood conduit. If this is to be accounted for by entrance loss, as the author suggests, assuming that there is no such loss in connection with the Englewood conduit and that $m = 0.01222$ for the Germantown conduit, then,

$$h = \frac{v_a^2}{2g}\left(1 + 0.1222\,\frac{546}{14.24} + C\right)$$

Taking an average test, as, for example, $h = 18.55$ and $v_a = 27.2$, then, $1.614 = 1 + 0.468 + C$, or $C = 0.145$; that is, 14.5% of the velocity head is absorbed as entrance loss, or the entrance loss of the Germantown conduit is $\frac{0.145}{0.468}$, or 31.0% of the friction head, or the entrance head gives a loss equal to about 169 ft. of conduit.

The two conduits are built alike; the area of the Germantown conduit is 2 by 91 sq. ft. and that of the Englewood is 2 by 108.5 sq. ft. It seems unlikely that the constant, m, should vary 20% when determined from measurements on conduits that are so nearly similar. The entrance head of 14.5% seems also excessive, especially in view of the author's opinion that the entrance loss at both conduits should be very small. The writer hopes the author will be able to throw some light on this matter. He may possibly have tests from other reservoirs that may be of assistance.

When the Chezy formula, $v = C\sqrt{r\,\frac{h_f}{L}}$, is used instead of Equation (2), Equation (3) takes the form:

$$h = \frac{v_a^2}{2g}\left(1 + \frac{2g}{C^2}\,\frac{L}{r}\right) \qquad (12)$$

Comparing Equation (3) with Equation (12), it will be seen that the only difference between them is the exponent of r. The coefficient, C, varies about

10% between the two conduits, while m, in Equation (3), varies about 20 per cent. One is hardly justified in concluding from tests on these two conduits of nearly equal dimensions and characteristics that the Chezy formula is the more nearly correct and, therefore, the more reliable, even if in this particular case it obviously fits the two sets of tests better than Equation (3). It would be a simple matter, of course, to determine the exponent, x, of r in Equation (3) in such manner that the coefficient, m, would be the same for the two conduits. This would give x equal to about 0.9, but it is equivalent to thrusting aside all other experimental evidence, which places x at about 1.168, as given by Unwin and referred to previously in this discussion.

As far as the writer knows, there is little information available concerning friction losses in large conduits for high velocities, and it seems, therefore, highly desirable that the meager data shall agree as closely as possible.

K. L. De Blois,* Jun. Am. Soc. C. E. (by letter).—This paper, with actual measurements made during recent high waters, should form an important addition to the Technical Reports of the Miami Conservancy District.

The author mentions that there was a question as to what point between the middle and the top of the conduit the head should be taken in computing the flows. This point cannot be determined theoretically since the outlet is neither an orifice nor an open channel. The conditions below the outlet, therefore, must govern the point to which the head should be taken, and this can be determined only experimentally. A wide variety of conditions may exist below the outlet, namely, the channel sides may be parallel or diverging and the bottom of the channel may slope down from the outlet at varying angles, or may drop off sharply at varying points below the outlet.

This question was made the subject of a thesis† at the Rose Polytechnic Institute, in June, 1922. The object of the experimental work was to find the law by which water flows through conduits, subsequently discharging into the air after passing through an open channel which is variable in length.

In the case of an orifice the flow is computed from the formula, $Q = c\,A\,\sqrt{2\,g\,h}$, in which, h is measured to the center of the section, or a trifle below the center if the head is relatively low compared to the height of the conduit. Fig. 21 (*a*) shows this condition. In the case of an open channel the head in the ordinary channel formulas is measured to the surface of the water. This condition is shown in Fig. 21 (*b*).

The object of these experiments was to show where the h in the formula, $Q = c\,A\,\sqrt{2\,g\,h}$, should be taken for different lengths of channel beyond the outlet, using a relatively low head. Lack of time permitted only the testing of a 3 by 3-in. square conduit with parallel channel sides and horizontal channel bottom. The drop-off lengths beyond the outlet were 1½, 3, 6, 12, 18, 24, and 30 in., respectively (see Fig. 21 (*c*) and Fig. 21 (*d*)).

* Asst. Engr., C. C. C. & St. L. Ry., Cincinnati, Ohio.

† "Experimental Study of the Flow of Water Through Orifices as Determined by Conditions Below the Outlet," by K. L. DeBlois and D. R. Spencer.

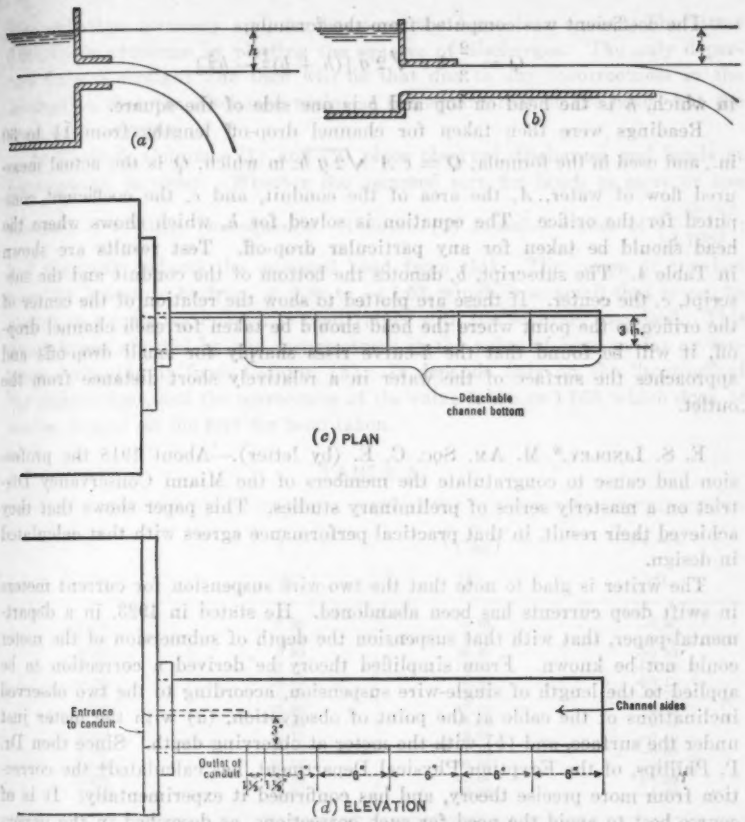

FIG. 21.—ARRANGEMENT OF TEST APPARATUS.

The tests were made to determine the relation between, Q, h, and the channel drop-off. Several readings were taken to determine the coefficient of the orifice which is the condition where the drop-off equals 0.

TABLE 4.—TEST RESULTS, 3 BY 3-INCH CONDUIT, HORIZONTAL CHANNEL.

Test No.	Drop-off, in inches.	Q, in cubic feet per second.	H_b, in inches.	H_c, in inches	h, in inches.	c, in inches $= H_c - h$.
1	0	0.259	5.23	3.73	$c =$	0.935
2	1½	0.258	5.67	4.17	3.63	0.54
3	3	0.247	5.80	4.30	3.33	0.97
4	6	0.232	5.60	4.10	2.94	1.16
5	12	0.250	6.18	4.68	3.40	1.28
6	18	0.236	5.79	4.29	3.04	1.25
7	24	0.244	5.97	4.47	3.25	1.22
8	30	0.242	6.16	4.66	3.20	1.46

The coefficient was computed from the formula,

$$Q = \frac{2}{3} b c \sqrt{2 g} [(h + b)^{\frac{3}{2}} - h^{\frac{3}{2}}]$$

in which, h is the head on top and b is one side of the square.

Readings were then taken for channel drop-off lengths from 1½ to 30 in., and used in the formula, $Q = c A \sqrt{2 g h}$, in which, Q is the actual measured flow of water, A, the area of the conduit, and c, the coefficient computed for the orifice. The equation is solved for h, which shows where the head should be taken for any particular drop-off. Test results are shown in Table 4. The subscript, b, denotes the bottom of the conduit and the subscript, c, the center. If these are plotted to show the relation of the center of the orifice to the point where the head should be taken for each channel drop-off, it will be found that the h-curve rises sharply for small drop-offs and approaches the surface of the water in a relatively short distance from the outlet.

E. S. LINDLEY,* M. AM. SOC. C. E. (by letter).—About 1918 the profession had cause to congratulate the members of the Miami Conservancy District on a masterly series of preliminary studies. This paper shows that they achieved their result, in that practical performance agrees with that calculated in design.

The writer is glad to note that the two-wire suspension for current meters in swift deep currents has been abandoned. He stated in 1923, in a departmental paper, that with that suspension the depth of submersion of the meter could not be known. From simplified theory he derived a correction to be applied to the length of single-wire suspension, according to the two observed inclinations of the cable at the point of observation, (*a*) with the meter just under the surface, and (*b*) with the meter at observing depth. Since then Dr. P. Phillips, of the Egyptian Physical Department, has calculated† the correction from more precise theory, and has confirmed it experimentally. It is of course best to avoid the need for such corrections, as described in the paper; but it is uncertain if that would be possible in an extremely swift deep current, such as that of the Indus in the Attock Gorge.

The writer is also glad to learn that the head for calculating the discharge of outlets has not been measured down to the centers of the orifices. Observations can be used to test the correctness of the assumed position of zero head. Unfortunately, the unavoidable error of observations of the kind made render a certain determination impossible without more readings than are published; but the writer wishes to demonstrate a method that he uses, which seems not to be generally known.

If discharges and water levels are plotted as in Fig. 5, the scale for levels must be small enough to accommodate a range of 35 ft. for Germantown and 45 ft. for Englewood. It is then too small for reading the quantity required

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† "An Experiment to Determine Corrections to Soundings," by P. Phillips, Egyptian Physical Dept., *Paper 18*, 1925.

with sufficient accuracy and, furthermore, the locus is a curve. The latter difficulty is overcome by plotting the squares of discharges. The only departure from a straight line then will be that due to any incorrectness in the assumption that the discharge varies as the square root of the head; but the small scale remains.

In Table 5, Columns (1) and (2) place observed discharges and heads at Germantown in order. Whether the assumed zero for heads is more or less incorrect is of little consequence. Column (3) includes values of $\left(\frac{Q}{1\,168}\right)^2$; and the next deducts the assumed head from this. The range of the last quantity is seen to be from + 0.80 to — 1.53, which is so small that it can be plotted to a scale big enough to read values with the accuracy required. This range is not affected by any incorrectness in the value of head as far as concerns deducting it from Column (3). It depends only on the "scatter" of the observations, and the correctness of the value taken as 1 168, which does, of course, depend on the zero for head taken.

TABLE 5.

Q. (1)	h. (2)	$\left(\frac{Q}{1\,168}\right)^2$ (3)	$d\sigma - h$ (4)
3 470	9.80	8.83	−0.97
3 653	10.15	9.78	−0.37
3 747	9.90	10.29	+0.39
4 196	12.15	12.91	+0.76
4 253	13.15	13.26	+0.11
4 617	16.40	15.62	−0.78
4 936	18.55	17.86	−0.69
5 038	19.45	18.61	−0.84
5 098	19.15	19.05	−0.10
5 365	21.65	21.10	−0.55
6 019	25.75	26.55	+0.80
6 294	28.65	29.04	+0.39
6 640	33.85	32.32	−1.53

The data of Columns (3) and (4) are plotted in Fig. 22. Because of the "scatter", each point is supplemented by a line measuring the effect of an error of 1% in the observed discharge. The data are so few that rejection of any one makes an appreciable difference to the result.

The "scatter" is such that there is considerable range of choice for the mean locus, with comparatively little difference of "probability". Two extreme alternatives are:

$$y = -0.27 \quad \ldots\ldots (13)$$

$$y = 0.08 - 0.0188\,x \quad \ldots\ldots (14)$$

The similar alternatives for Englewood are:

$$y = -0.31 \quad \ldots\ldots (15)$$

$$y = 0.35 - 0.038\,x \quad \ldots\ldots (16)$$

Equations (13) and (15) entail a change of the zero point for heads, without change of the multiplication factors. Equations (14) and (16) involve a

change of multiplication factor as well, calculated by substituting $\left(\frac{Q}{1\ 168}\right)^2$ for x, and $\left(\frac{Q}{1\ 168}\right)^2 - h$ for y.

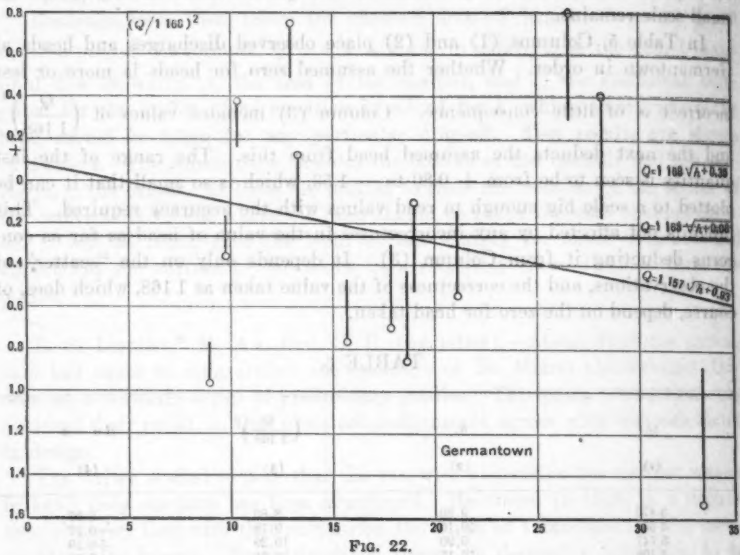


FIG. 22.

If, now, h is written for the head measured down to the crown of the outlet, the author's formulas and the writer's two alternatives become (Table 6):

TABLE 6.

	Germantown.	Englewood.
Author (Table 1).....	$Q = 1\ 168\ \sqrt{h + 0.35}$	$Q = 1\ 404\ \sqrt{h + 0.35}$
Equations (13) and (15).	$Q = 1\ 168\ \sqrt{h + 0.08}$	$Q = 1\ 404\ \sqrt{h + 0.04}$
Equations (14) and (16).....	$Q = 1\ 157\ \sqrt{h + 0.43}$	$Q = 1\ 378\ \sqrt{h + 0.70}$

Equations (14) and (16) have the advantage that they bring the values of Chezy's C down to 156 and 168, making them more nearly equal, and Kutter's n to perhaps more probable values; but they have the disadvantage that the zero points for head show more difference than seems likely from the similarity of form of the outlets. This only emphasizes the fact that the data are insufficient to determine the factors correctly.

C. S. JARVIS,* M. AM. SOC. C. E. (by letter).—Among the benefits that have accrued from the Miami Conservancy Project is the renewed or strengthened confidence in the correctness of conclusions based on comprehensive research, broadened views, and experimental methods, even when they are

* Highway Engr.; Chf. of Hydrographic Section, U. S. Bureau of Public Roads, Washington, D. C.

applied to such elusive subjects as probable rainfall and run-off, routing of floods, and advisable protective measures. The world-wide search for scientific data, the analytical and graphical rendering of that information, the well-conceived hydraulic laboratory experiments, and the concurrent opinion of experts in their respective fields, all led to the adoption of designs which have withstood the six-year test in full accordance with expectations. So effectively have these reservoirs and channels operated that their actual service in flood protection has frequently if not generally been under-estimated. Without regulative measures or channel improvements one or more of the recent floods would undoubtedly have caused serious damage to property in some localities adjoining the Miami River; the new growth and prosperity of the entire valley would have been retarded; and property values in the district would have declined, according to careful estimates, in an amount exceeding the total cost of the protective system.

It is reassuring to learn that the discharge through the conduits exceeds the estimated values by 14%, and that the open channels have some excess capacity. These results were to be anticipated when the errors or uncertainties were thrown to the side of safety. No fear need be entertained on account of this surplus capacity. If the storms and run-off should ever actually equal those assumed for purposes of design (which are regarded as beyond the probable limit of severity attainable in the Miami Valley), the influence of the increased outflow from the reservoirs would be reflected in decreased depth of storage at the peak of the flood. However, there is the remote probability of temporary obstructions, as from poles or uprooted trees that may lodge across a conduit intake after the most violent storms, in spite of the precautionary measures now in vogue for removing fallen timber or cutting it into short lengths. A moderate excess in discharge capacity should be regarded as an advantage on this project.

Any one who views the administrative work in progress in the Miami Valley must be impressed with the orderly methods used to obtain new observations and measurements, as well as to portray and interpret the information. Outstanding among the impressions received by the writer during an inspection trip over the principal structures was the idea of security where the situation had long been precarious, and of confidence, enterprise, and prosperity which had followed in the wake of positive flood protection. While the main problems seem to have been solved satisfactorily, there are details of administration, maintenance, operation, construction, and minor improvements which receive constant attention. In due time, these should form the basis for additional descriptive and analytical contributions to the fund of practical engineering knowledge.

E. W. Lane,* M. Am. Soc. C. E. (by letter).—This paper is especially interesting to the writer since he had the pleasure of working on the project during most of the design period. Although the measurements at the Englewood and Germantown conduits indicate a combined discharge of about 24 500 sec-ft., instead of the 22 000 assumed in the design, it

* Engr., U. S. Bureau of Reclamation, Denver, Colo.

would be a mistake to conclude that the same percentage of error would result in the discharges of the conduits at the other three dams. At the latter, the "conduits" are simply openings through the bottom of the ogee spillway section and are, therefore, of short length. One of the advantages of this type of outlet is that the discharge is nearly independent of the roughness of the concrete. The friction term, $\frac{2\,g\,L}{C^2\,r}$, in the formula, $Q = c\,A\,\sqrt{2\,g\,h}$, in which, $c = \sqrt{\frac{1}{1 + \frac{2\,g\,L}{C^2\,r}}}$, is small for small values of L, especially when the conduits are large, since both C and r are relatively large.

An error in estimating the roughness of the concrete in such conduits, therefore, has little effect. If the roughness at these dams was $n = 0.0101$ instead of $n = 0.0130$ as designed, the discharge would be increased only about 1% at Lockington and about 0.5% at Taylorsville and Huffman. With this assumption for the flow from the Lockington, Taylorsville, and Huffman Basins and the outflows indicated by the measurements from the Germantown and Englewood Basins, the flow at Dayton and Hamilton would exceed the design values by about 2% only, and, therefore, need raise no doubts of the ability of the system to take care of the flood for which it was designed.

The location shown in Fig. 10 is especially favorable for checking the formulas for determining the difference of elevation of the water surface around bends.* The mean difference of elevation at the two sides on March 21, 1927, as shown by data furnished to the writer, was 0.091 ft. Let d = the difference in water surface elevation at the two sides of the stream; V = the mean velocity = 4.06 ft. per sec.; b = breadth of stream = 6.37 ft.; g = acceleration due to gravity = 32.2 ft. per sec. per sec.; and R = the radius of curvature of the center of the stream = 4 093. Then, approximately, $d = \frac{V^2\,b}{g\,R} = 0.080$ ft.

Assuming a parabolic distribution of velocities with zero velocity at the edges, by dividing the channel into six equal strips and computing the difference for each strip and totaling them, a result of 0.096 is obtained. Considering the difficulties of accurate measurement of the water-surface elevations in the field, these seem to be satisfactory checks. The computed difference for the flow for which the channel was designed, is about 1.3 ft.

Charles H. Paul,† M. Am. Soc. C. E. (by letter).—Full-scale results, under actual operating conditions, are most valuable to the engineer who is faced with a problem of design. Had information of this kind been available at the time the Miami flood-control project was designed, much time, labor, and expense would have been saved to the District.

There is much of value in this paper which unfortunately cannot be indicated by the title. Some items, especially, should be mentioned, as fol-

* "Hydraulics of the Miami Flood Control Projects," by S. M. Woodward, M. Am. Soc. C. E., p. 272.

† Managing Director, Dayton Industrial Assoc.; Cons. Engr., Dayton, Ohio.

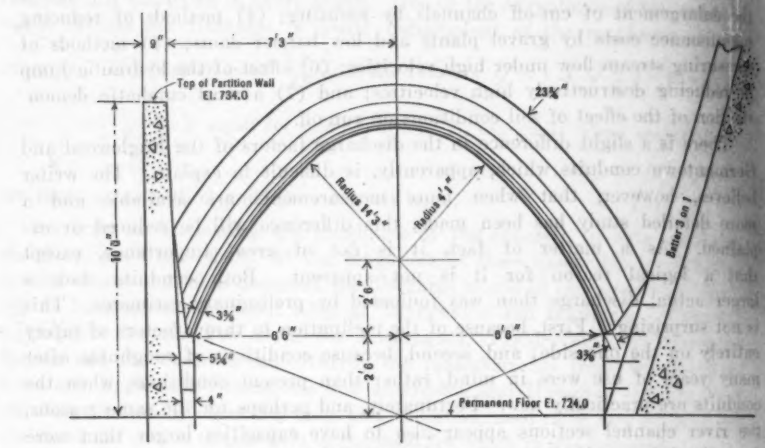
lows: (1) Actual capacities of large conduits under heads up to 40 ft. or more; (2) actual values of n for improved river channels under various conditions; (3) practical demonstrations of the economical methods of securing the enlargement of cut-off channels by scouring; (4) methods of reducing maintenance costs by gravel plants and low barrier dams; (5) methods of measuring stream flow under high velocities; (6) effect of the hydraulic jump in reducing destructively high velocities; and (7) a most emphatic demonstration of the effect of soil conditions on run-off.

There is a slight difference in the discharge factors of the Englewood and Germantown conduits which, apparently, is difficult to explain. The writer believes, however, that when more measurements are available and a more detailed study has been made, this difference will be reduced or explained. As a matter of fact, it is not of great importance, except that a logical reason for it is not apparent. Both conduits show a larger actual discharge than was indicated by preliminary estimates. This is not surprising: First, because of the inclination to throw factors of safety entirely on the one side; and, second, because conditions of roughness after many years of use were in mind, rather than present conditions, when the conduits are practically new. Fortunately, and perhaps for the same reasons, the river channel sections appear also to have capacities larger than were determined by preliminary estimates. If these comparisons hold for the higher discharges, that is, if both the conduits and the improved river channels have larger discharges than were estimated, that in itself will tend to maintain the balance of the system as a whole. If for any reason it is desired to reduce the capacities of the conduits either temporarily or permanently, this can be done easily and at small expense by simply raising the floors at the inlet ends as much as may be desired.

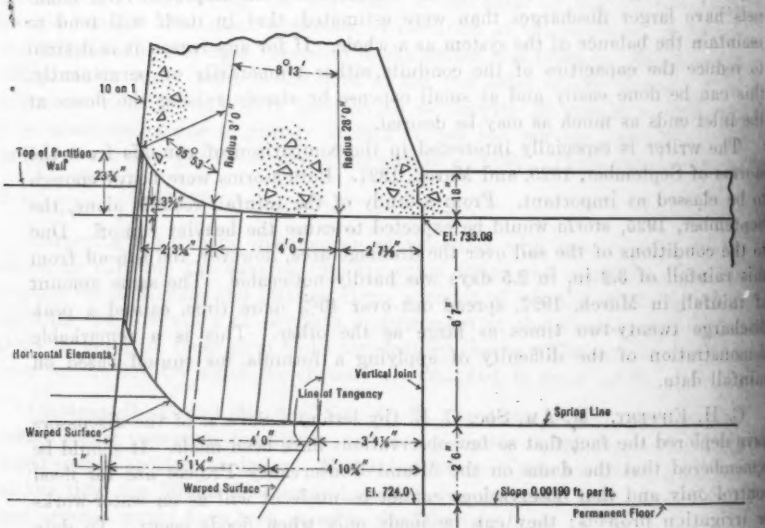
The writer is especially interested in the comparison of run-offs from the storms of September, 1925, and March, 1927. Both storms were heavy enough to be classed as important. From a study of the rainfall records alone, the September, 1925, storm would be expected to cause the heavier run-off. Due to the conditions of the soil over the drainage area, however, the run-off from this rainfall of 3.2 in. in 2.5 days was hardly noticeable. The same amount of rainfall in March, 1927, spread out over 40% more time, caused a peak discharge twenty-two times as large as the other. This is a remarkable demonstration of the difficulty of applying a formula for run-off based on rainfall data.

C. H. EIFFERT,* M. AM. SOC. C. E. (by letter).—Several of the discussors have deplored the fact that so few observations have been made. It should be remembered that the dams on the Miami Conservancy Project are for flood control only and that observations cannot be made at will as on water-works or irrigation projects; they can be made only when floods occur. To date (1929) it has not been possible to make any satisfactory observations at the Taylorsville and Huffman Dams, where the largest discharges occur. The conduits at these dams have been full only twice, and one of these occurrences was at night, so that sufficient data are not available.

* Chf. Engr., The Miami Conservancy Dist., Dayton, Ohio.



(*a*) FRONT VIEW



(*b*) LONGITUDINAL CROSS SECTION

FIG. 23.—DETAILS OF BELLMOUTH ENTRANCES TO CONDUITS.

FIG. 24.—DISCHARGE AT GERMANTOWN, LOOKING UP STREAM TOWARD DAM, FLOOD OF FEBRUARY 25–26, 1929.

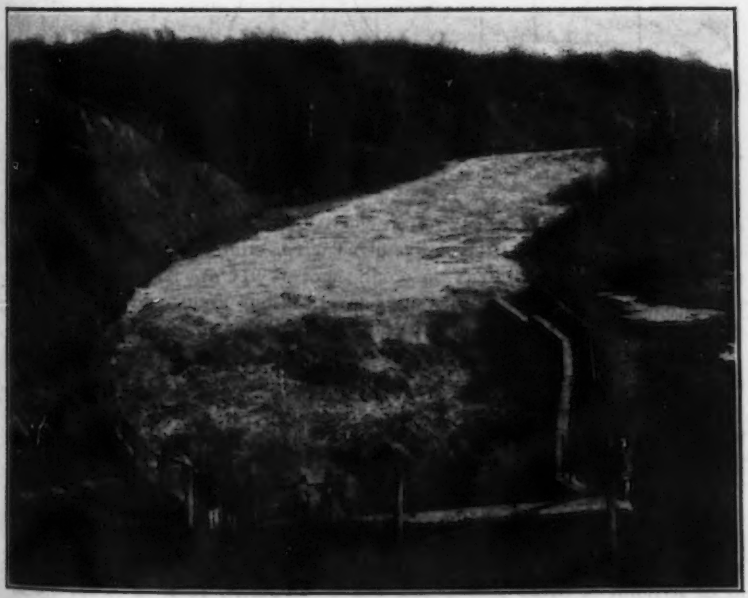
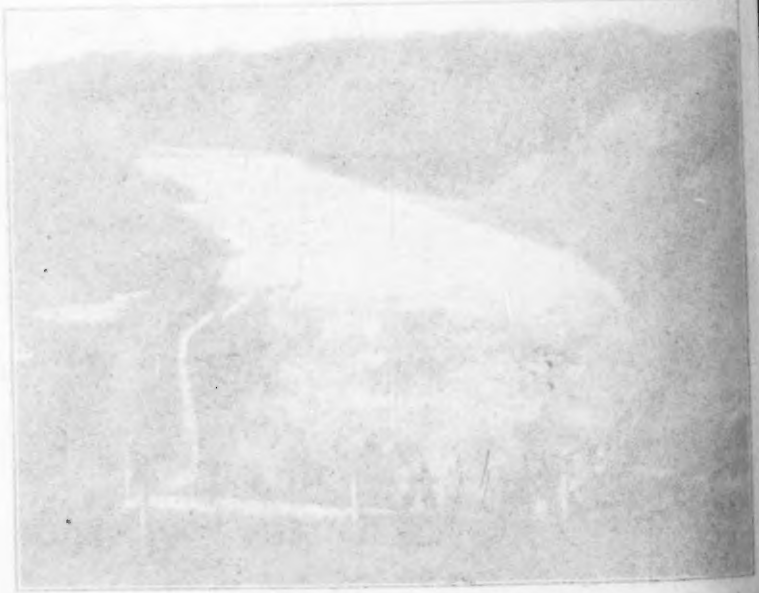


FIG. 25.—DISCHARGE AT GERMANTOWN, LOOKING DOWN STREAM, FLOOD OF FEBRUARY 25–26, 1929.



Mr. Smith states that the use of a value of n for the conduit design which is greater than that observed in the completed structures is in this case quite the opposite of a factor of safety. When the conduits were designed it was known that 0.13 was too great for smooth new concrete. It was not known, however, what amount of deterioration might take place in time, thus increasing the actual value of n. It was also recognized that if the conduits proved too large, that is, if the maximum discharges were greater than could be handled safely by the channels in the cities, the conduits could easily be reduced in size; whereas, if they had proved too small, to increase them in size would have been very difficult if not impossible. Discharges from these conduits of less than the design calls for would cause higher water levels in the reservoirs than were anticipated and so would reduce the factor of safety of the dams.

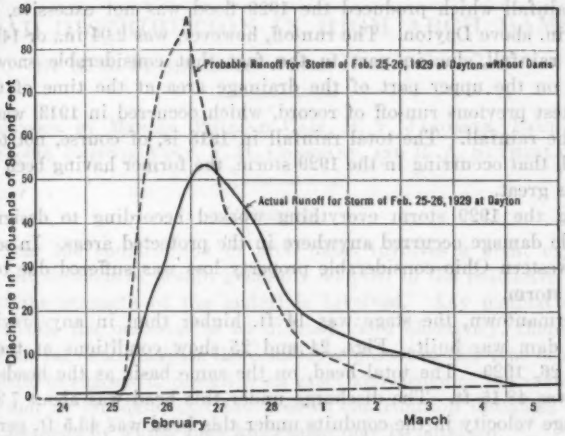


FIG. 26.—REDUCTION OF PEAK RUN-OFF FOR STORM OF FEBRUARY 25 AND 26, 1929, AT DAYTON, OHIO.

Mr. Smith also applies the difference in the designed and actual discharges at Englewood to the conduits at the Taylorsville and Huffman Dams. The conduits at Englewood are 712 ft. long; those at Taylorsville and Huffman are less than 50 ft. in length and are much larger in diameter. A calculation for these dams will show that the effect of the roughness factor is practically negligible. This was shown by Mr. Lane in his discussion.

The conduits are provided with a carefully designed bellmouth entrance as stated by Mr. Smith. The scale of the drawing in Fig. 6 is too small to show this feature. A larger illustration is provided in Fig. 23. While the entrance loss may be greatly reduced or practically eliminated by this design, the writer does not believe that it has been reduced to zero. If there is some entrance loss, it has not been considered in the calculation. The determined values of n would be still further reduced if the entrance losses could be calculated.

The writer is unable at present to explain the reason for the difference in the values of n at Germantown and Englewood, which is referred to in Mr. Jakobsen's discussion. About the same number of measurements were

taken at each location and exactly the same methods were used at both places. As stated in the paper the writer believes that there is a possibility of these values being affected by the entrances to the conduits. By this is meant the concrete entrance channels and not the bellmouth entrances to the conduits themselves.

Since the paper was written a larger flood has occurred (February 25 and 26, 1929) than any on which data were previously given. It would have been nearly as large as that of 1898 (the next to the largest of which there is an accurate record), at Dayton, without the retarding basins and it would have exceeded the 1898 flood at Hamilton in the lower part of the valley. There was storage at all the dams, that at Germantown, Taylorsville, and Huffman being greater than in any previous flood. The total storage, in all five basins, however, was only about 10% of the combined capacities of the basins.

The rainfall which produced the 1929 flood was not excessive, averaging only 1.80 in. above Dayton. The run-off, however, was 2.04 in., or 14% greater than the rainfall, due in part to the fact that considerable snow and ice remained on the upper part of the drainage area at the time of the storm. The greatest previous run-off of record, which occurred in 1913, was equal to 91% of the rainfall. The total rainfall in 1913 is, of course, not to be compared with that occurring in the 1929 storm, the former having been five times or more as great.

During the 1929 storm everything worked according to design and no appreciable damage occurred anywhere in the protected areas. In some cities in Southwestern Ohio considerable property loss was suffered due to flooding from this storm.

At Germantown, the stage was 14 ft. higher than in any previous flood since the dam was built. Figs. 24 and 25 show conditions at the dam on February 26, 1929. The total head, on the same basis as the heads given in Table 2, was 48.15 ft. The discharge under this head was about 7 900 sec-ft. The average velocity in the conduits under this head was 43.5 ft. per sec. and, where the water enters the natural channel below the outlet works, the average velocity was 8.6 ft. per sec. Fig. 26 shows the reduction of the peak flood run-off at Dayton for the storm of February, 1929. It is similar to Fig. 14 except that the reduction is greater.

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Paper No. 1719

LOAD DISTRIBUTION IN HIGH ARCH DAMS*

By R. A. Sutherland,† Assoc. M. Am. Soc. C. E.

With Discussion by Messrs. William Cain, B. F. Jakobsen, Paul Bauman, A. Floris, Fredrik Vogt, F. H. Constant, and R. A. Sutherland.

Synopsis

Every year the construction of arch dams involves a great expenditure of money, but methods of design generally adopted in the past have not made full use of the strength of the materials involved. Any method of design that will enable the material to be utilized more effectively, therefore, will effect large savings and increase the confidence placed in this type of structure. A prerequisite to successful design is an exact knowledge of the various forces acting on and in the dam, and the present contribution discusses the principles and methods by which such knowledge may be obtained.

On account of the complexity of the factors involved, previous methods of load analysis were based on many assumptions which aimed at reducing these factors to a form susceptible to mathematical treatment. The principal assumptions are herein reviewed, particularly those relating to the so-called cantilever action. The assumption most strongly attacked is that which regards the arch as being uniformly loaded, and a powerful method of correctly determining the actual non-uniform beam and arch loading is explained.

The methods proposed are based primarily on beam and arch deflections and by means of charts and simplified procedure the application of these methods is made comparatively simple and accurate, and dependent on the minimum number of arbitrary assumptions. When the actual loading on beams and arches under any given set of conditions is known, the resultant stresses may readily be determined, and such modifications in design may be made as are required to enable the dam to carry the load most effectively.

* Published in April, 1928, *Proceedings*.

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The lines along which improvements in design may be made, are then suggested.

A satisfactory method of arch dam design requires a more complete knowledge of actual temperature variations in concrete, of its elastic properties, and of the effect on the dam of the elastic properties of the foundation rock. Such knowledge will be obtained mainly from measurements and observations on existing dams. Apart from the incompletely known effect of these factors, arbitrary assumptions in the past have led to errors and should be avoided.

Load Distribution

A number of published investigations on this subject have appeared during the last few years (see Bibliography). Of these, some are so highly mathematical as to be beyond the scope of many engineers, whereas others, in order to obtain simplicity, have introduced assumptions which militate against their accuracy. It is the writer's hope that his method may indicate, perhaps, the lines on which a convenient and accurate analysis may be evolved. To avoid covering elementary ground, a general acquaintance with recently published investigations is assumed, in particular with that* of Fred A. Noetzli, M. Am. Soc. C. E. The latter is a very clear statement of the whole problem, and includes comparatively simple methods for the analysis of load distribution which are extremely useful in the preliminary design of an arch dam. To promote uniformity Mr. Noetzli's symbols have been used as much as possible. It may be mentioned that the work of Professor Camillo Guidi, of Turin, Italy, is of great value to any engineer interested in arch analysis, and should be better known in other countries. Guidi's analysis, although developed primarily from an academic standpoint, are direct and may readily be adapted for practical use, as is shown by their extensive adoption in his own country. For instance, his methods were used in the design of such important multiple-arch dams as that at Tirso, which is probably the largest of its kind in the world.

The arch dam may be regarded as a curved plate of varying thickness and generally of irregular outline, restrained only along a part of its periphery, and subjected to non-uniform loads due to water pressure, temperature, and concrete shrinkage. The single feature favorable to a stress analysis, as against the case of structures, such as bridges or buildings, is that the total water load is known with absolute accuracy. This load, however, is not always the most important one in producing stresses, and certain assumptions have to be made in regard to temperature and shrinkage loads.

Most arch dams have been designed on the basis of the "cylinder formula", in which it is assumed that the dam is composed of a number of horizontal arch slices, each consisting of a part of the wall of a complete thin cylinder. It has always been recognized that this assumption is far from representing actual conditions, and to compensate for it, the stresses allowed in design are generally lower than in other types of structures using the same materials. In time, this tacit confession of ignorance must be replaced by more complete and more exact knowledge of the distribution of load in an arch dam, and it seems reasonable to expect that such knowledge will permit the use of

* *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 1.

higher stresses and enable modifications in dimensions to be made, which will result in economy of material and, at the same time, in a greater degree of confidence in the safety of such structures.

Before proceeding to the examination of actual stress distribution in an arch dam, it will be well to examine how far the cylinder theory may be assumed to hold good, even considering that the dam may be divided legitimately into a number of independent arch slices. The equation:

$$t = \frac{P_c r_u}{f} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

is based on the assumption of a perfect and complete cylinder subjected to uniform pressure, the thickness of which is negligible in comparison with the radius. Errors are obviously introduced in the following respects:

(1) The cylinder may not be perfectly circular. A flattened cylinder will obviously endure greater stresses because of bending moments in the planes of the major and minor axes. Arched dams are bound to vary slightly from a true circular shape, but errors due to this cause are probably not great, and of course will be introduced by any method of design.

(2) The ratio of thickness to mean radius in an arch may be as much as 0.72 (Shoshone). It is small only in the upper portion of a dam where direct arch load is small. A high ratio does not, of course, effect the average stress on any section, but causes an inequality between the stresses on the intrados and the extrados.

(3) Probably the greatest error is introduced by the fact that, on account of restraint at the sides of the gorge, all parts of the arch ring are not equally free to deflect radially, as they would be in a complete circular ring. This restraint at once introduces bending moments which have the effect, not only of altering the distribution of stress at any given section, but also of producing a variation between the average stresses at different sections. The analysis of a fixed ended arch under uniform load is given in Appendix I, and, by means of Figs. 11 to 13, the actual stresses at four points in the arch may be found at once for any given conditions. These points are the extradosal and intradosal points of the crown and abutment sections, respectively. It should be mentioned that in this discussion the arch is assumed to be fixed at the abutments. This fixation, although perhaps not absolutely in accord with actual conditions, is considered to be more correct than Mr. Noetzli's assumption of a two-hinged arch, for example, which is inconsistent with the assumption of a fixed ended cantilever.

It will be evident from the foregoing that the cylinder formula, except for small thin arch dams, is unworthy of the confidence frequently placed in it. Most attempts to evolve an alternative basis of design have been along the lines of complicated mathematical theory in which, in many cases, the dam was assumed to be confined between vertical abutments and to have a horizontal base. William P. Creager, M. Am. Soc. C. E., expresses this opinion clearly,* stating further that various compensating and cumulative effects, such as stresses due to temperature changes, moisture content, etc.,

* "Masonry Dams," N. Y., John Wiley & Sons, 1917, p. 154.

"are sufficient to nullify practically the basic assumptions of such investigations." To the writer, however, the fact that certain assumptions must be made with regard to temperature, shrinkage, etc., appears to be no warrant for evading the investigation of the loads caused in a dam in the fulfillment of its primary purpose, namely, that of retaining water. The load due to the water is known with absolute accuracy.

The object of the present paper is to make some small contribution toward:

(*a*) The determination of the distribution of the primary or water load, and of the resultant stresses.

(*b*) The determination, with suitable assumptions, of the effect caused by secondary loads due to temperature, shrinkage, etc.

(*c*) The indication of possible variations in design and construction with a view to a better utilization of material and consequent economy.

The fundamental distribution of load in an arch dam has been stated with great clearness in most of the papers mentioned in the Bibliography, and will be clear from the following considerations. An arch can take its full load only if it is perfectly free to deflect. The crest sections of an arch dam approximate this condition of freedom, but at points nearer the base a greater restraint comes into play, until at the base itself no deflection is possible, and this has led to the conception of the "vertical cantilever"; that is, the dam is assumed to be composed of a series of imaginary vertical strips,* each resting on a series of imaginary horizontal arch strips, constituted of the same material. Among the arch strips, the degree of freedom is a maximum at the crest and decreases to zero at the base. The result is, obviously, that the lower arches take less and less of the total load, which is progressively transferred to, and taken by, the cantilever strips. Assuming the cantilevers to be fixed at the base, a considerable bending moment will be developed there. If on the other hand the cantilevers (which now become beams) are assumed to be hinged or freely supported at the base, no bending moment can be developed there, and the effect of the beams is then merely to transfer load from the more heavily loaded arch strips to those less heavily loaded. Bending moments will occur in the beam itself, but not at the base. Both these cases are dealt† with very clearly by the late R. Shirreffs, M. Am. Soc. C. E., on the basis, however, of assumptions "some of which", Creager states,‡ "are impossible, and others of which are unusual."

It will be well at the outset to examine the legitimacy of the various assumptions made by previous writers. The principal of these are:

1.—The site section is of a rectangular or other regular shape.
2.—The up-stream and the down-stream faces in a vertical section are rectilinear.
3.—The geometrical shape of every vertical section is similar.
4.—The horizontal cross-section of the vertical cantilever is rectangular.
5.—The load distribution on any horizontal arch slice is uniform.
6.—Shear deflection may be neglected.
7.—Young's modulus is constant.

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* See p. 1192, Fig. 1.

† *Transactions*, Am. Soc. C. E., Vol. LIII (1904), p. 155.

‡ "Masonry Dams," p. 154.

Assumption 1.—Practically all authorities assume vertical gorge sides and horizontal floor, which typifies the average dam site very remotely. For example, Noetzli dismisses variations from this assumed site section by stating* that: "In general it will be desirable to investigate also the sections at about the quarter-points of the highest arches." The writer wishes to emphasize, not only the desirability of investigating the sections at about the quarter-points of the highest arches, but the absolute necessity of doing so if a correct load distribution is to be obtained.

Cross-sections of the sites of several of the largest dams in the world are shown in Fig. 1. The approximate dimensions are also given. The Shoshone and the Pathfinder Dams are the only ones which approach a rectangular shape. The error of the assumption in the majority of cases is very evident.

However, the dimensions of most masonry dam sites can be reduced to a mathematical formula of the type, $x = Ay^m$. This fact makes it possible to obtain simple expressions for the quantity of masonry in any type of dam built in such a site.†

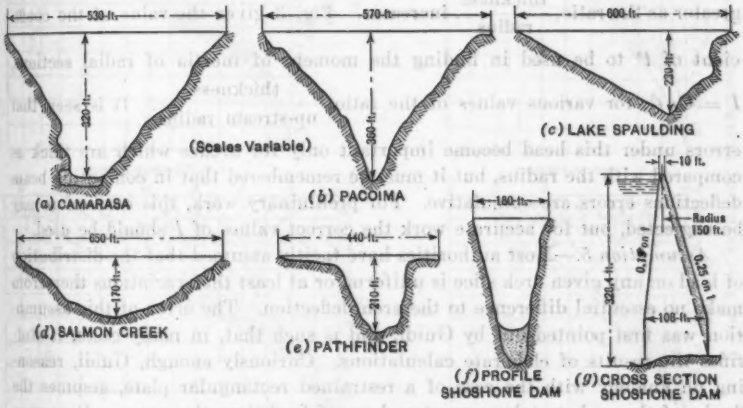

FIG. 1.—SITE SECTIONS OF VARIOUS DAMS.

Assumption 2.—Most writers consider the dam to be of triangular section, which is the theoretical shape for both gravity and arched dams. Many small dams, such as the Barren Jack Creek Dam, in New South Wales, and Las Vegas, in New Mexico, are practically triangular in section, with a slight thickening at the crest. In most cases, however, the section is trapezoidal, as shown in Fig. 1 (*g*). The extra material more than the theoretical amount is shown stippled, and although this may appear to be an inconsiderable addition it represents, toward the crest, a relatively large increase on the theoretical thickness, and causes a considerable divergence from the results evolved on the triangular profile. Noetzli finds the deflection for a trapezoidal cantilever by first computing it for a triangular and for a parallel-sided section, and taking a pro-

* *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 25.
† *Commonwealth Engineer* (Australia), June, 1928.

portionate figure between the two. This also introduces errors. Fig. 2 shows the deflection at any point of a trapezoidal cantilever, supporting a water load, for various values of the ratio, $\frac{\text{crest thickness}}{\text{base thickness}}$. The error of assuming a linear relationship between this ratio and crest deflection is evident.

In the case of the constant-angle dam, the section is not even trapezoidal. Shirreffs* has indicated how allowance can be made for this variation from the assumed section, but his complicated equations are of little use in any practical case. The arithmetical method explained by the writer covers any actual case that may arise in practice.

Assumption 3.—In the case of dams of homocentric curvature, any cross-section is, of course, similar in shape. In constant angle dams, however, the section varies according to its position. The method of analysis, to be correct, should take cognizance of this fact.

Assumption 4.—The magnitude of errors introduced by the assumption that horizontal cross-sections of the cantilever are rectangular, becomes greater as the ratio, $\frac{\text{thickness}}{\text{radius}}$, increases. Fig. 3 gives the value of the coefficient of t^3 to be used in finding the moment of inertia of radial sections, $I = C_I\, t^3$, for various values of the ratio, $\frac{\text{thickness}}{\text{up-stream radius}}$. It is seen that errors under this head become important only for arches which are thick as compared with the radius, but it must be remembered that in computing beam deflections errors are cumulative. For preliminary work, this correction may be neglected, but for accurate work the correct values of I should be used.

Assumption 5.—Most authorities have tacitly assumed that the distribution of load on any given arch slice is uniform, or at least that variations therefrom make no essential difference to the arch deflection. The error of this assumption was first pointed out by Guidi, and is such that, in many cases, it stultifies the results of elaborate calculations. Curiously enough, Guidi, reasoning on analogy with the case of a restrained rectangular plate, assumes the flanks of the arch to take a greater share of load than the crown. It appears probable that the reverse is the case.

A small variation from uniform distribution of load on an arch will cause a difference in crown deflection that is out of all proportion to the magnitude of such variation. Non-uniform arch load may best be taken account of by the use of influence curves. (See Appendix II.)

Assumption 6.—The assumption has generally been made that shear deflection is negligible. As every engineering student knows, shear deflection is important only in beams or arches the thickness of which is great compared with their length. It may sometimes be safely neglected for the central cantilever sections of an arch dam, or for the highest arch slices, but considering the ease with which it may be computed, it is believed to be desirable in all cases to allow for it. The neglect of shear deflection will generally cause an error of from 5 to 15 per cent.

* *Transactions*, Am. Soc. C. E., Vol. LIII (December, 1904), p. 155.

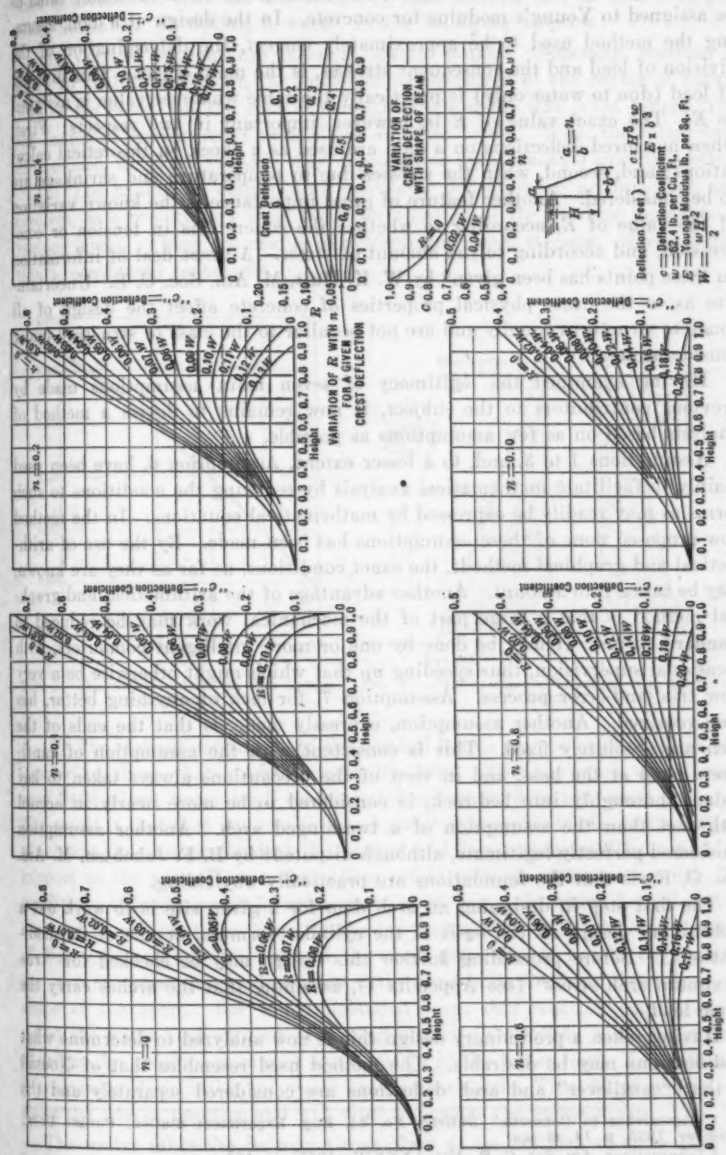

FIG. 2.—CANTILEVER DEFLECTION CURVES.

Assumption 7.—Much discussion has taken place as to the exact value to be assigned to Young's modulus for concrete. In the design of a dam, assuming the method used to be approximately correct, the determination of the division of load and the consequent stresses, is the main object. This division of load (due to water only) is practically the same whatever value is assigned to E. The exact value of E is, however, important in two respects: First, when measured deflections on a dam are used as a check on theoretical calculations; and, second, when the stresses due to temperature and shrinkage are to be considered. Another feature of great importance is the known variation of the value of E, according to whether the concrete is in tension or compression, and according to the amount of stress. A great deal of information on these points has been given* by W. K. Hatt, M. Am. Soc. C. E. Uncertainties as to the exact physical properties of concrete affect the design of all concrete structures equally and are not peculiar to the class of structure under consideration.

Having examined the legitimacy of seven main assumptions made by previous contributors to the subject, it now remains to devise a method of analysis based on as few assumptions as possible.

Assumptions 1 to 5, and, to a lesser extent, Assumption 6, have been used mainly to facilitate mathematical analysis by reducing the conditions to such forms as may readily be expressed by mathematical equations. In the method now proposed none of these assumptions has been made. By the use of arithmetical and graphical methods, the exact conditions, as far as they are known, may be taken into account. Another advantage of the arithmetical and graphical method is that a large part of the mechanical work may be reduced to standard forms. It may be done by one or more intelligent computers, with occasional supervision, thus speeding up that which might otherwise be a very slow trial-and-error process. Assumption 7, for lack of anything better, has been retained. Another assumption, expressly made, is that the ends of the arch are absolutely fixed. This is consistent with the assumption of cantilevers fixed at the base, and in view of the precautions always taken to key a dam thoroughly into bed-rock, is considered to be more nearly in accord with fact than the assumption of a two-hinged arch. Another assumption considered perfectly legitimate, although disputed† by B. F. Jakobsen, M. Am. Soc. C. E., is that the foundations are practically unyielding.

The first step in designing an arch dam for a given site is to work out a preliminary design on the basis of the cylinder formula, or one of its modifications.‡ Before proceeding further this design may be checked for "true maximum arch stress" (see Appendix I), assuming that the arches carry the whole load.

Having taken a preliminary design this is now analyzed to determine what modifications may be desirable. The method used resembles that of Noetzli in that "cantilever" and arch deflections are considered separately and the

* "Researches in Concrete," *Bulletin No. 24*, Eng. Experiment Station, Purdue Univ., November, 1925, p. 14, *et seq.*

† *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 102.

‡ See "The Control of Water," by the late Philip A Morley Parker, M. Am. Soc. C. E., p. 403.

load is divided so that the deflections by each method are, as nearly as possible, the same. It differs, however, in the following respects:

(1) Vertical sections are taken at several points. Generally, sections at every 10° will be sufficient, the angles being measured at the center of curvature of the crest. In the case of constant-angle dams this point differs from the center of curvature of arch slices at other levels. With the latter type of dam also, the flank sections may approach or even exceed a gravity section.

(2) The method of determining cantilever deflections is arithmetical, and involves, as a preliminary step, the determination of the bending moments at various levels. These are useful in determining beam stresses. Shear deflection is considered separately.

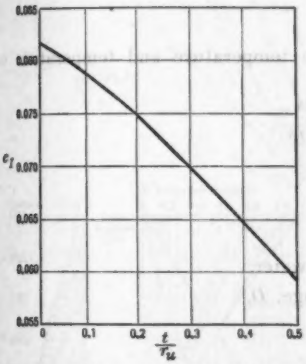

FIG. 3.—MOMENTS OF INERTIA FOR RADIAL SECTIONS.

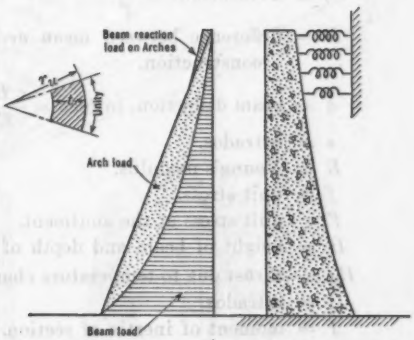

FIG. 4.—DIAGRAMMATIC LOADING FOR VERTICAL BEAM.

(3) The statical conditions of the vertical beams may be considered as very similar to those that probably actually exist. The top arch slices carry very little direct water load, especially if the water does not reach the crest. The result is that they act as a series of yielding supports to the vertical slices, which are thus beams fixed at the bottom and freely supported by a yielding support at the top. Hence, the writer refers to these strips as beams. The conditions of loading may be represented as in Fig. 4, in which helical springs are used to represent the thrust of the top arches. From the deflection curves of Fig. 2, it will be seen that, if the crest deflection is not to be negative, the amount of reaction load possible at the crest is limited, and depends on the shape of the beam. For a parallel-sided beam this reaction cannot exceed 0.2 of the water load without producing negative deflection, while for a beam of triangular section, it cannot exceed 0.075 of the water load. In both cases a triangular distribution of water load is assumed. This is, of course, not quite the case in practice and will alter somewhat the values given.

The writer takes the Salmon Creek Dam as an example to show that the method herein proposed is quite general. The example given merely illustrates the general method and is not a complete analysis. In order to produce

exact agreement between arch and cantilever deflections possibly a half dozen separate trials would be required.

Notation

The following notation is used in the paper:

a = top thickness of beam (Fig. 2).

A = a constant $= \frac{x}{y^m}$.

α = coefficient of linear expansion applied to arch material.

b = base thickness of beam.

c = beam deflection coefficient $= \frac{\delta E b^3}{H^5 w}$.

C_I = a coefficient $= \frac{I}{t^3}$.

D = difference between mean arch temperature and temperature of construction.

δ = beam deflection, in feet $= \frac{c H^5 w}{E b^3}$.

e = extrados.

E = Young's modulus.

f = unit stress.

f' = unit stress at the abutment.

H = height of beam and depth of water.

H_D = thrust due to temperature change, D.

i = intrados.

I = moment of inertia of section.

m = a constant.

$n = \frac{a}{b}$.

p_e = external pressure due to water load = 6.24 H lb. per sq. ft.

r = mean radius of arch.

r_u = radius of up-stream face.

R = "crest-reaction" load on beam.

t = thickness of arch (Fig. 7).

w = weight of a unit cube of water.

W = total water load on beam.

x = width of canyon at a height, y, above bed-rock.

y = height above bed-rock to the point where x is measured.

Distribution of Water Load

Sections of the proposed dam, all taken radially through the center of curvature of the crest, are set out, and the water-load diagram is drawn against each one. The water load on the central section is now divided into that part which is to be taken by beam action, and that which is to fall on the arches (Fig. 5(*c*)). This may be done merely by judgment or by Noetzli's preliminary trial method, using the curves of Fig. 2 for beam deflections, and the arch deflection coefficients of Fig. 13 (Appendix I). In using Fig. 2, the non-

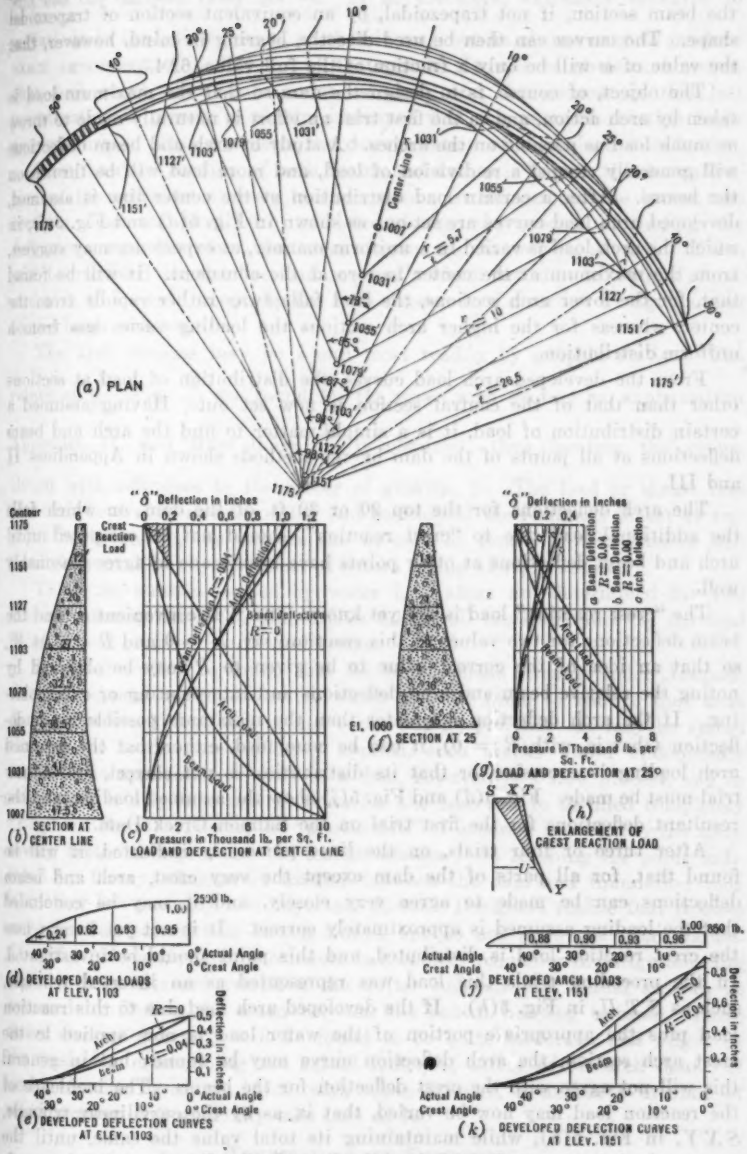

FIG. 5.—DESIGN OF SALMON CREEK DAM.

triangular water load should be replaced by an equivalent triangular load, and the beam section, if not trapezoidal, by an equivalent section of trapezoidal shape. The curves can then be used directly, bearing in mind, however, that the value of w will be only a fraction of the full value, 62.4.

The object, of course, is to design the dam so that the maximum load is taken by arch action, and in the first trial an effort is naturally made to throw as much load as possible on the arches. A study of arch and beam deflections will generally compel a re-division of load, and more load will be thrown on the beams. After a certain load distribution at the center line is assumed, developed arch load curves are set out as shown in Fig. 5(d) and Fig. 5(j), in which the arch load is varied in a uniform manner, as experience may suggest, from the maximum at the center to zero at the abutment. It will be found that, for the lower arch sections, the load falls away rather rapidly from the center, whereas for the higher arch sections the loading varies less from a uniform distribution.

From the developed arch load curves, the distribution of load at sections other than that of the central section is now set out. Having assumed a certain distribution of load, it is a simple matter to find the arch and beam deflections at all points of the dam by the methods shown in Appendices II and III.

The arch deflections for the top 20 or 30 ft. of the dam, on which falls the additional load due to "crest reaction", should not be computed until arch and beam deflections at other points have been made to agree reasonably well.

The "crest reaction" load is not yet known, and it is convenient to find the beam deflections for two values of this reaction, say, $R = 0$ and $R = 0.04\ W$, so that an idea of the correct value to be given to R may be obtained by noting the relative beam and arch deflections and interpolating or extrapolating. If the arch deflection is greater than the maximum possible beam deflection (that is, with $R = 0$), it can be concluded, either that the assumed arch loading is excessive, or that its distribution is not correct, and a new trial must be made. Fig. 5(d) and Fig. 5(j) show the assumed loading, and the resultant deflections for the first trial on the Salmon Creek Dam.

After three or four trials, on the lines previously indicated, it will be found that, for all parts of the dam except the very crest, arch and beam deflections can be made to agree very closely, and it may be concluded that the loading assumed is approximately correct. It is not yet known how the crest reaction load is distributed, and this point should be investigated. In the preceding trials this load was represented as an inverted triangle, such as $S\ T\ U$, in Fig. 5(h). If the developed arch load due to this reaction load plus the appropriate portion of the water load is now applied to the crest arch section, the arch deflection curve may be found; but in general this will not agree with the crest deflection for the beams. The incidence of the reaction load may now be varied, that is, as by the curvilinear triangle, $S\ X\ Y$, in Fig. 5(h), while maintaining its total value the same, until the crest arch deflections are brought into line with the beam deflections. It must be remembered that the crest arch section subtends the greatest angle

and has the highest value for $\frac{r}{t}$. It is, therefore, extremely sensitive to variations in load distribution, so that it may not be always easy to produce exact agreement of deflections at that point. It will be obvious, however, that such exact agreement is not essential, and the purpose of this last investigation is merely to obtain assurance that the assumed reaction loads are reasonable. This completes the determination of water-load distribution.

Once the exact loading of both beams and arches is known, the determination of the resultant stresses becomes easy. In the case of the beams, the bending moments have been tabulated during the process of finding the deflections, and, therefore, the beam stresses can be written at once. These will be modified by the "no-load stresses", which are due to weight alone, and also, to a small extent, by the arch stresses in a direction at right angles to the beam stresses.

The arch stresses may be found most readily by graphic statics. The values of $\frac{M}{P\,r}$ and $\frac{H}{P}$ are found for the given arch loading as explained in Appendix II, and from these $\frac{M}{H\,r}$ is obtained, thus locating the crown thrust with reference to the center of gravity, G. The load or thrust line can then be drawn, and the value and position of the resultant thrust at any point being known, the stresses can at once be found. These stresses will be slightly modified by the effect of others acting at right angles.

Thus, the stresses caused by water load alone are determined for two extreme assumptions: (1) The arches taking all the load; and (2) the load being shared between beams and arches.

It will generally be found that the beam tensile stresses under Assumption (2) appear to be excessive, which has led some to believe that arch dams must crack, but this is not borne out by practice. It may be stated that if the loads were suddenly applied, such cracks would undoubtedly occur. The application of load in a dam, however, is a slow process, and in some cases it may take several years for the reservoir to be filled completely (Elephant Butte). Hence, the "plastic flow" of concrete is evidently the main saving factor that prevents the tensile stresses from reaching the point indicated by theory. Imperfect fixation between the dam and foundation rock is another but probably less important factor. Designers realize that stresses must lie between certain known limits, but do not know the exact values. The effect of plastic flow of concrete is a subject requiring investigation, and until such results are available, the writer suggests that actual stresses be assumed by finding the mean between the known limiting stresses. These mean stresses generally will be found to be quite moderate, and might be the basis for determining the actual factor of safety of an arch dam. If undue tension is shown, even with mean stresses, a moderate amount of reinforcement will generally take care of it.

Temperature and Shrinkage

Temperature and shrinkage can be classed together since the effect of shrinkage is identical with that produced by a drop of temperature. For in-

stance, in the latest Italian Government regulations for dams, shrinkage is to be allowed for as a fall of temperature of 18° Fahr. For any given combined temperature and shrinkage effect (Fig. 12, Appendix I) the values of the stresses at the four critical points of the arch can be found if it is assumed that beam action does not restrain the arches. With the same assumption, the arch deflection curve may be traced, either from Fig. 13 which applies to the crown deflection only, or by the use of the deflection influence curves, Fig. 15, by regarding the temperature change (for the purposes of arch deflection only) as equivalent to a uniform water load such as would produce a cylinder stress equal to $E \alpha D$.

This conception of the "equivalent water load" may be misleading as a means of finding arch stresses. In considering deflection and stresses the arch was assumed to deflect to its final position in two steps. First, the ends being assumed free to slide, the whole arch was allowed to shrink. In this position, a certain deflection has occurred, due to temperature, which is completely analogous to that due to water load. This is the "Hauptsystem" of the German writers.* In the case of temperature shrinkage, however, the arch is unstressed, while under water load it is subjected to uniform cylinder stress. The second step, that of moving the abutment back to its original position by the application of a horizontal thrust, is identical in regard to its effect on both deflection and stress. The result is that for each step the two cases are analogous as regards deflection, but not as regards stress.

In the particular case considered, the center line section of Salmon Creek Dam, a temperature change varying uniformly from 10° Fahr. at the base to 20° Fahr. at the crest, has been assumed. The unrestrained arch deflection curve is shown in Fig. 6(*a*). The "equivalent water load" is also shown. It is obvious that beam action will restrain the arch deflection just as it did in the case of water load, and it might be thought that it would be sufficient to analyze the distribution of this hypothetical water load by the same procedure as was applied in the case of the actual water load which, however, is not correct. In the case of water load, arches and beams deflect until at a certain point each is in equilibrium under the influence of its own share of load and the resisting forces called into play thereby. At a few points in the dam the load on the arch and that on the beam may happen to be equal, but, in general, this is not so. In the case of temperature, however, the deflecting loads on the beams are produced directly by the deflection of the arches, and since action and reaction are equal and opposite, the actual loading on beam and arch is at all points equal.

What actually happens may be analyzed with the help of Fig. 6. Assume that the arch and beam strips are perfectly independent, and allow the temperature variation to occur. The beams will remain unmoved, but the arches will move to a position, such as $L\ M\ N$. The "equivalent water load" producing this deflection is purely imaginary. Both beams and arches are now in a position of rest, which may be taken as a vertical, $L\ Q$, for the beams, and $L\ M\ N$ for the arches. Now, allow the interaction between beams and

* "Die Staumauern," by Kelen, 1926.

arches to come into play. These will now come together until they coincide at some position, such as $L\ P\ O$. The beam deflection produced is represented by the difference between the vertical, $L\ Q$, and the line, $L\ P\ O$, while the arch deflection produced is represented by the difference between $L\ M\ N$ and $L\ P\ O$. Both deflections have been produced by equal but opposite loads, and these loads are real, being the system of forces generated by the interaction of arches and beams.

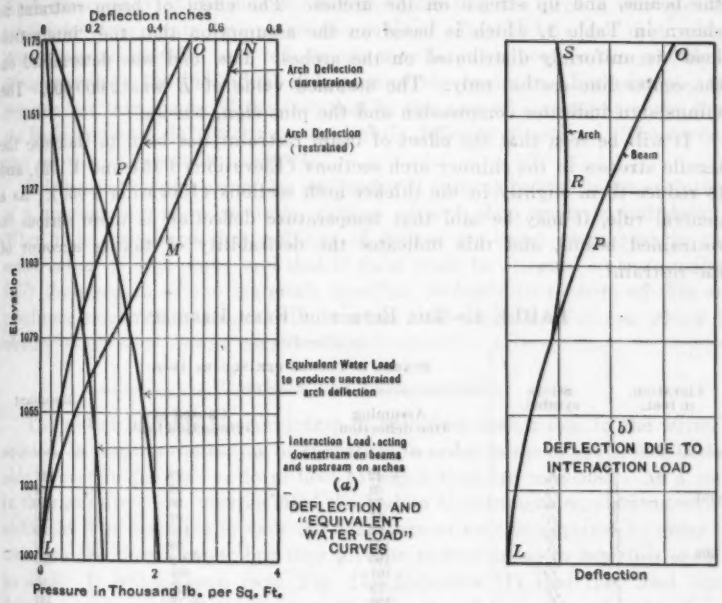

FIG. 6.—TEMPERATURE DEFLECTION CURVES.

Representing arch and beam deflections in the usual way, as in Fig. 6(*b*), it is required to find the real load, which, if applied equally to beams and arches, would cause the former to deflect to the position, $L\ P\ O$, and the latter to the position, $L\ R\ S$. The load which would produce this arch deflection can be determined without trouble, and it then simply remains to check whether or not this same load would produce the required beam deflection. If not, a new trial deflection line, $L\ P'\ O'$, must be taken and the process repeated. After two or three trials the required load can be found.

To determine the stresses is then not difficult. In the case of the beams the only load acting is the real load just found, and the bending moments have already been recorded in the course of the deflection calculation, so that the stresses can be found immediately. In the case of the arches it is necessary to find first the stresses caused by the free deflection (to $L\ M\ N$), and these can be determined at once for the four critical points of the arch from Fig. 12, or for any other point by considering the effect of the thrust, H_D,

which can be readily evaluated. These arch stresses must then be corrected for the effect of the stresses caused by the real "interaction load" which has just been found, and which may be regarded as a water load (although its variation with depth will not, in general, be linear). These second stresses are such as sometimes to ameliorate, and sometimes to aggravate, the effect of the first stresses. In the case considered (drop of temperature), the interaction load shown is to be regarded as a water load acting down stream on the beams, and up stream on the arches. The effect of beam restraint is shown in Table 1, which is based on the assumption that the "interaction load" is uniformly distributed on the arches. The load was determined on the center line section only. The assumed value of E was 2 000 000. The minus sign indicates compression and the plus sign, tension.

It will be seen that the effect of beam restraint has been to increase the tensile stresses in the thinner arch sections (Elevations 1 151 and 1 103), and to reduce them slightly in the thicker arch sections (Elevation 1 055). As a general rule, it may be said that temperature deflection is more serious in restrained beams, and this indicates the desirability of taking account of the restraint.

TABLE 1.—THE EFFECT OF BEAM RESTRAINT.

Elevation, in feet.	Stress symbol.	Stress, in Pounds per Square Inch.		Resultant.
		Assuming free deflection.	Caused by interaction load.	
1 151	f_o	— 42	79	37
	f_i	39	53	92
	f_o'	70	36	106
	f_i'	— 66	92	26
1 103	f_o	— 76	115	39
	f_i	109	29	138
	f_o'	190	— 8	182
	f_i'	—167	161	— 6
1 055	f_o	— 79	125	46
	f_i	156	— 19	137
	f_o'	265	— 72	193
	f_i'	—205	193	— 12

COMBINED EFFECT OF WATER LOAD AND UNIFORM TEMPERATURE VARIATION

The simplest way to investigate the combined effect of water load and uniform temperature variation is to consider each one separately and combine the resultant stresses. In this manner it is possible to find, for each part of the dam, the maximum stress occurring under any possible set of conditions, and provision should be made either to sustain this stress, or to reduce it by appropriate alterations in the design.

The most important cases are: (1) Full water load and temperature drop; and (2) no water load and temperature rise. It has been shown that temperature drop, combined with the effect of beam restraint, will cause a fairly general state of arch tension. Hence, under water load, part of the arch compression will be cancelled by this initial tension, and the arch will be more lightly stressed than at the normal temperature. At the same time the

beams will be more heavily loaded. It should be noted that the effect of temperature drop is not merely to throw more of the water load on the beams, since the load diagram now contains extraneous loads not comprised within the water load diagram.

General Discussion

The writer has intentionally confined himself to discussing general principles, since many of the details of their application are already very clearly treated in other contributions to the subject. It will be clear that many details may be modified and refinements may be introduced in the procedure as suggested by the individual judgment of the engineer. For example, no mention has been made of the fact that in the case of a constant angle dam the beam and arch deflections are in slightly different directions. The correction for this is generally negligible.

It is hoped that the principles outlined will eventually lead to the evolution of a quicker, but more accurate, method of arch dam analysis and design. When it is considered that millions of dollars are spent every year in the construction of arch dams, and that if these could be designed so as to utilize fully the strength of the materials specified, probably 20 to 40% of this expenditure might be saved, it will be clear that the subject is one which is worthy of the very closest consideration.

Proposals for Improvements

The widest field for improvements in arch dam design lies, in the writer's opinion, in varying the radius of curvature in order to make the arches better able to sustain the non-uniform load to which they are subjected. As a step in the right direction, may be cited the Italian Government regulations which state that it is desirable to vary the curvature as may be required in order to maintain the thrust axial; but they give no indication as to how this should be done. It will be seen from Fig. 17 (Appendix II) that the "load line" departs considerably from the geometrical axis of the arch, and the object of all arch design is to make the two as nearly coincident as possible. This may be done by appropriately varying the curvature of the arch, except that it vitiates at once the relatively simple methods of analysis discussed in this paper. Recourse must then be had to arithmetical and graphical methods of analysis, which in the present case are very laborious. In the case of a dam, the designer is, of course, limited in varying the curvature by the necessity of avoiding overhang. Another proposal investigated by the writer for counteracting the effects of beam action was that of giving the dam a slight overhang in an up-stream direction, at the same time adding a moderate amount of hoop steel to insure stability with reservoir empty. The object was to create sufficient residual compression at the up-stream heel to cancel the beam tension. This is practically equivalent to removing beam action altogether, and thus enabling the dam to act purely as an arch. Such a proposal is not advantageous in the case of a single arch dam, but might, in certain cases, be worthy of investigation. The Coolidge Dam,* of the multiple-dome type,

* *Engineering News-Record*, May 27, 1926, p. 865.

appears to take advantage of this residual compression at the up-stream heel, by reason of the beam section being convex up stream.

It has been seen that the excess arch thickness near the crest has a beneficial effect in reducing beam stresses, and the writer considers that additional concrete placed in the crest is a sound investment.

Acknowledgments

The writer acknowledges his great indebtedness to the work of Professor Guidi, whose method of analysis has been followed very closely in Appendices I and II. He also wishes to thank Mr. R. J. Spowart for the very careful preparation of most of the diagrams and for considerable assistance in the computations, and Mrs. R. A. Sutherland for assistance in preparing the paper for publication.

APPENDIX I

Notation

The following notation is used:

α = coefficient of linear expansion of arch material.
B = coefficient for determining crown deflection under uniform load (see Fig. 13).
c = coefficient for finding true arch stress from cylinder stress.
d = distance from the center line (Fig. 10) of the elastic center of the half arch.
d_x = distance of the antipole, Y, from Axis x-x (with respect to the elastic ellipse of the half arch (see Equation (59)).
D = difference between mean arch temperature and temperature of construction.
δ = crown deflection.
e = extrados.
E = Young's modulus.
f = unit stress.
f' = unit stress at the abutments.
g = elastic weight (in general).
dg = elastic weight of the arch element.
g' = elastic weight of the half arch.
G = coefficient of rigidity.
h = rise of arch.
Δh_p = total crown deflection, due to water load.
Δh_D = total crown deflection, due to a uniform variation of temperature, D, from the temperature of construction.
Δh_H = crown deflection, due to the effect of arch thrust.
Δh_k = crown deflection, or change of arch rise, due to uniform arch shortening (neglecting the effect of arch thrust).
H_D = thrust due to temperature change, D.
H_p = horizontal thrust due to water pressure.

i = intrados.

I = moment of inertia of unit arch slice $\left(=\frac{t^3}{12}\right)$.

I_x = moment of inertia of the elastic arch with respect to x-x axis.

I_X = moment of inertia of the elastic arch with respect to X-X axis.

K = constant depending only on the dimensions of the arch section (see Equation (22)).

l = chord of arch ($A\,B$, Fig. 7) measured from center to center of haunches.

L = length of mean arc.

ΔL = length of arch element.

M = bending moment.

p_e = external pressure due to water load.

ρ = radial semi-axis of the ellipse of elasticity of the arch element.

ρ_1 = tangential semi-axis of the ellipse of elasticity of the arch element (Fig. 8).

r = mean radius of arch.

r_u = radius of up-stream face.

t = thickness of arch (Fig. 7).

T = coefficient for finding stresses due to uniform temperature change.

τ = temperature.

τ_e = difference between extradosal temperature and temperature at which arch was sealed (temperature of construction).

τ_i = difference between intradosal temperature and temperature of construction.

$\Delta\tau$ = difference of temperature between faces.

x = distance from Y-axis of the center of gravity of the arch element.

x-x = axis through the center of gravity of the mean arc and parallel to the chord (see Fig. 7).

X-X = axis parallel to x-x through the center of curvature.

χ = coefficient relating to the distribution of shear over a cross-section, and depending only on the shape of the section.

y = distance from the x-x axis of the center of gravity of the arch element.

y' = distance from the Axis x-x of the antipole of x-x with respect to elastic ellipse of the arch element.

y_X = distance from Axis X-X of the center of gravity of the arch element.

$y_{X'}$ = distance from Axis X-X of the antipole of X-X with respect to the elastic ellipse of the arch element.

Z = section modulus.

ϕ = angle measured at the center, O (Fig. 8), from the crown to the arch element.

$\Delta\phi$ = differential angle, ϕ.

ϕ_0 = one-half the central angle measured at the center, O (Fig. 7).

Stress Analysis of an Arch Subjected to Uniform Water Pressure*

A horizontal circular arch strip is taken, with unit height and uniform thickness, and is loaded on the extradosal face by a uniform water load. In such an arch $\left(\text{assuming the ratio, } \frac{\text{radius}}{\text{thickness}}, \text{ to be great}\right)$ the pressure line would correspond with the geometrical arch axis, if the arch were indeformable, and in this case the load on every radial section would reduce to a normal uniform compression of the value, $p_e\, r_u$, or of the value, $p\, r$, in which, r is the mean radius and (see Fig. 7),

$$p = p_e \frac{r_u}{r} \qquad (2)$$

Actually, the arch is shortened by the load to which it is subjected, and the effect of this is evaluated in the following manner. If the abutment exerted simply a thrust, $p\, r$, normal and central, and did not otherwise restrain the section, A, each arch element, ΔL, would be shortened by the amount,

$$d(\Delta L) = \frac{p\, r}{E\, 1\, t}\, \Delta L \qquad (3)$$

Hence, the chord, l, would be shortened by the amount,

$$\Delta l = \frac{p\, r}{E\, t}\, l \qquad (4)$$

or, this amount of translation to the right, without rotation, would have to be given to the section, A. Since, however, Section A is effectively immovable, it must develop a supplementary reaction capable of annulling the displacement mentioned, which reaction, according to the elastic theory, has for its line of action the axis, x-x, passing through the elastic center of gravity, G,

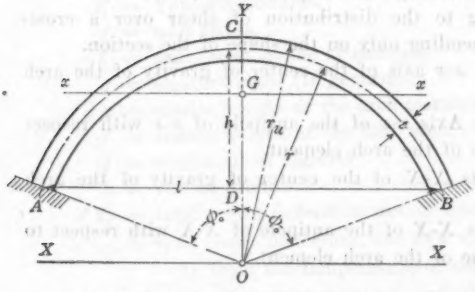

Fig. 7.—Arch Dimensions.

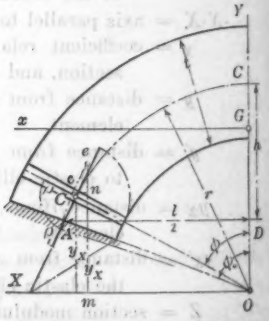

Fig. 8.—Arch Stress Theory.

* Adapted from "Statica delle dighe per laghi artificiali," by Prof. Camillo Guidi, Royal Polytechnic Coll., Turin, Italy, pub. by Vincenzo Bona, Turin, 1921.

of the arch, and parallel to the chord, $A\,B$. This reaction is directed outward (that is, negative) and its value is given by,

$$H_p = \frac{\frac{p\,r}{E\,t}\,l}{\frac{I_z}{}} \quad \ldots\ldots(5)$$

Now, the elastic weight of the arch element is given by,

$$dg = \frac{dL}{E\,I} \quad \ldots\ldots(6)$$

Hence, dg is proportional to dL, and, therefore, the elastic center of gravity, G, of the arch corresponds with the center of gravity of its geometrical axis. Referring to Fig. 7:

$$I_X = \frac{12}{E\,t^3}\int y\,y'\,d\,L \quad \ldots\ldots(7)$$

This is the moment of inertia with respect to the axis, X, passing through the center of curvature. Then, transferring to Axis x:

$$I_x = \frac{12}{E\,t^3}\left(\int y_X\,y_{X'}\,d\,L - (\text{line } O\,G)^2\int d\,L\right) \quad \ldots\ldots(8)$$

Fig. 8 represents the half arch, with an exaggerated ratio of radius to thickness for the sake of clearness.

For the arch element, ΔL (Fig. 8):

$$\rho^2 = \frac{1}{12}\,t^2 \quad \ldots\ldots(9)$$

and,

$$\rho_1{}^2 = \frac{(\Delta L)^2}{12} + \chi\,\frac{E}{G}\,\rho^2 \quad \ldots\ldots(10)$$

Taking $\frac{E}{G} = \frac{5}{2}$, and remembering that, for a rectangular section, $\chi = \frac{6}{5}$:

$$\rho_1{}^2 = \frac{(\Delta L)^2}{12} + 3\cdot\rho^2 \quad \ldots\ldots(11)$$

When ΔL becomes infinitesimal,

$$\rho_1{}^2 = 3\,\rho^2 = \frac{t^2}{4}$$

For any arch element (Fig. 8):

$$y_X = r\cos\phi \quad \ldots\ldots(12)$$

$$y_{X'} = (\text{line } m\,n) + (\text{line } n\,e) = (\text{line } O\,c\cos\phi) + (\text{line } c\,e\sin\phi)$$

$$= \left(r + \frac{\rho^2}{r}\right)\cos\phi + \frac{\rho_1{}^2}{r\cot\phi}\sin\phi = \left(r + \frac{t^2}{12\,r}\right)\cos\phi + \frac{t^2}{4\,r}\,\frac{\sin^2\phi}{\cos\phi} \quad \ldots\ldots(13)$$

Also,

$$d\,L = r\,d\,\phi \quad \ldots\ldots(14)$$

and,

$$(\text{line } O\,G) = \frac{r\cdot\sin\phi_0}{\phi_0} \quad \ldots\ldots(15)$$

Hence,

$$\int y_X\, y_{X'}\, d\,L = 2\,r^3\left[\left(1+\frac{t^2}{12\,r^2}\right)\int_0^{\phi_0}\cos^2\phi\,d\,\phi+\frac{t^2}{4\,r^2}\int_0^{\phi_0}\sin^2\phi\,d\,\phi\right]$$

$$=r^3\left[\left(1+\frac{t^2}{12\,r^2}\right)\left(\phi_0+\frac{\sin 2\,\phi_0}{2}\right)+\frac{t^2}{4\,r^2}\left(\phi_0-\frac{\sin 2\,\phi_0}{2}\right)\right]\ldots.(16)$$

Also,

$$(\text{line } (O\,G)^2)\int d\,L = 2\,r^3\,\frac{\sin^2\phi_0}{\phi_0}\ldots\ldots\ldots\ldots\ldots\ldots(17)$$

Hence,

$$I_x=\frac{12\,r^3}{E\,t^3}\left[\left(1+\frac{t^2}{12\,r^2}\right)\left(\phi_0+\frac{\sin 2\,\phi_0}{2}\right)\right.$$

$$\left.+\frac{t^2}{4\,r^2}\left(\phi_0-\frac{\sin 2\,\phi_0}{2}\right)-\frac{2\sin^2\phi_0}{\phi_0}\right]\ldots\ldots\ldots\ldots\ldots(18)$$

From geometry (Fig. 8):

$$\phi_0=\frac{L}{2\,r}\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(19)$$

$$\frac{\sin 2\,\phi_0}{2}=\sin\phi_0\cos\phi_0=l\,\frac{r-h}{2\,r^2}\ldots\ldots\ldots\ldots\ldots\ldots(20)$$

$$\frac{2\sin^2\phi_0}{\phi_0}=\frac{l^2}{r\,L}\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(21)$$

Hence,

$$I_x=\frac{6\,r^2\,l}{E\,t^3}\left[\frac{L}{l}+\frac{r-h}{r}-\frac{2\,l}{L}+\frac{t^2}{6\,r^2}\left(\frac{2\,L}{l}-\frac{r-h}{r}\right)\right]\ldots\ldots(22)$$

Calling the expression in square brackets, K_0, this reduces to,

$$I_x=\frac{6\,K_0\,r^2\,l}{E\,t^3}\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(23)$$

Equation (5) now reduces to,

$$H_p=p\,r\,\frac{t^2}{6\,K_0\,r^2}\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(24)$$

On account of this outward supplementary reaction, the line of pressure is displaced outward above the axis, x, and inward below this axis. The algebraic sum of the areas between the new pressure line and the geometric axis of the arch is almost exactly zero.

Maximum Stresses Due to Water Load

The sections of the arch most heavily stressed are those at the crown and at the abutment. Assuming the trapezium law to be correct (Fig. 8):

$$f_e=-\frac{p\,r}{t}-\frac{6\,H_p\left[(\text{line } G\,C)-\dfrac{t}{6}\right]}{t^2}\ldots\ldots\ldots\ldots\ldots(25)$$

$$f_i = -\frac{p\,r}{t} + \frac{6\,H_p\left[(\text{line } G\,C) + \frac{t}{6}\right]}{t^2} \quad \ldots\ldots (26)$$

$$[\text{line } G\,C] = r\left(1 - \frac{\sin\phi_0}{\phi_0}\right) = r\,\frac{L-l}{L} \quad \ldots\ldots (27)$$

Substituting the value of H_p from Equation (24):

$$f_e = -p\left[\frac{r}{t} + \frac{1}{K_0}\left(\frac{L-l}{L} - \frac{t}{6\,r}\right)\right] \quad \ldots\ldots (28)$$

$$f_i = -p\left[\frac{r}{t} - \frac{1}{K_0}\left(\frac{L-l}{L} + \frac{t}{6\,r}\right)\right] \quad \ldots\ldots (29)$$

also,

$$f_e' = -\frac{p\,r}{t} + \frac{6\,H_p}{t^2}\left[(\text{line } G\,D) + \frac{t}{6}\cos\phi_0\right] \quad \ldots\ldots (30)$$

$$f_i' = -\frac{p\,r}{t} - \frac{6\,H_p}{t^2}\left[(\text{line } G\,D) - \frac{t}{6}\cos\phi_0\right] \quad \ldots\ldots (31)$$

Since (Fig. 8),

$$(\text{line } G\,D) = (\text{line } G\,O) - (\text{line } D\,O) = \frac{r\sin\phi_0}{\phi_0} - r\cos\phi_0$$

$$= r\left(\frac{l}{L} - \frac{r-h}{r}\right) \quad \ldots\ldots (32)$$

therefore,

$$f_e' = -p\left\{\frac{r}{t} - \frac{1}{K_0}\left[\frac{l}{L} - \frac{r-h}{r}\left(1 - \frac{t}{6\,r}\right)\right]\right\} \quad \ldots\ldots (33)$$

$$f_i' = -p\left\{\frac{r}{t} + \frac{1}{K_0}\left[\frac{l}{L} - \frac{r-h}{r}\left(1 + \frac{t}{6\,r}\right)\right]\right\} \quad \ldots\ldots (34)$$

The value determined from Equation (34) is always the greatest of the four stresses evaluated. Hence, Noetzli's assumption of a two-hinged arch errs not only qualitatively, but quantitatively, in that it assumes the crown stresses to be the deciding factor.

Stresses Due to Temperature Variation

In general, the up-stream and down-stream faces of a dam will not be at the same temperature, nor will they be at the temperature of construction. Hence, stresses are introduced due both to the mean variation from the temperature of construction and to the difference of temperature between the two faces. Calling these τ_e and τ_i for the extrados and intrados, respectively, and assuming a linear variation between the two, it is safe to assume that the values, τ_e and τ_i, have been reached by: (1) A uniform variation of temperature equal to $\frac{\tau_e + \tau_i}{2} = D$ from the temperature of construction; and (2) a local variation of $\frac{\tau_e - \tau_i}{2}$ at the extrados, and $\frac{-\tau_e + \tau_i}{2}$ at the intrados. Assuming a linear variation between τ_e and τ_i may be quite legitimate for thin

arches, such as small dams, or multiple-arch dams, but will not in general be even approximate for large arch dams.

For example, supposing both τ_e and τ_i represent rises in temperature, Fig. 9 shows (to an exaggerated scale) the effect on a small arch element. Assuming first that one abutment of the arch is free, the terminal face of the arch element, EF, would assume the position, $E'F'$, in which, $EE' = \alpha \tau_e \Delta L$ and $FF' = \alpha \tau_i \Delta L$.

This deformation consists of two parts: (1) A translation, $d(\Delta L)$, due to the general rise of temperature, $\frac{\tau_e + \tau_i}{2}$; and (2) a rotation expressed by:

$$d(\Delta \phi) = \frac{\alpha(\tau_e - \tau_i)}{t} \Delta L = \frac{\alpha \Delta \tau \Delta L}{t} \quad \ldots\ldots (35)$$

The first deformation taking place over the whole arch, would produce a translation of the free abutment of the dam by a distance,

$$\alpha \frac{\tau_e + \tau_i}{2} l = \alpha D l$$

Since the abutment is effectively immovable, the thrust exerted along the axis, x, in an inward direction, has the value,

$$H_D = \frac{\alpha D l}{I_x} \quad \ldots\ldots (36)$$

and substituting the value of I_x, Equation (23),

$$H_D = \frac{t^3}{6 K_0 r^2} E \alpha D \quad \ldots\ldots (37)$$

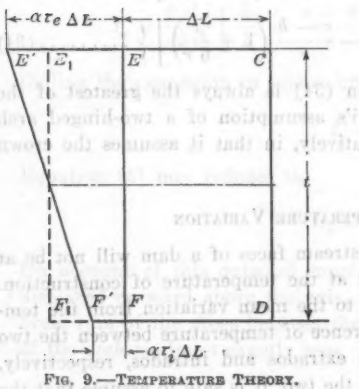

FIG. 9.—TEMPERATURE THEORY.

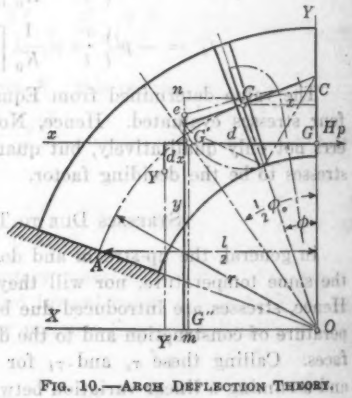

FIG. 10.—ARCH DEFLECTION THEORY.

The second deformation, due to the local temperature variations of the two faces, would cause the end of the arch, if free—and hence the elastic center of gravity, G, which is always considered to be rigidly connected thereto—to undergo a rotation,

$$\Delta \phi = \alpha \Delta \tau \sum \frac{\Delta L}{t} \quad \ldots\ldots (38)$$

Hence, the point, G, would be translated a distance, the co-ordinates of which are,

$$\Delta x = \alpha \Delta \tau \sum \frac{\Delta L}{t} y \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (39)$$

$$\Delta y = \alpha \Delta \tau \sum \frac{\Delta L}{t} x \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (40)$$

The term, t, being constant, may be taken outside the summation sign, and since G coincides with the center of gravity of the arch axis,

$$\sum y \Delta L = \sum x \Delta L = 0 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (41)$$

hence,

$$\Delta x = \Delta y = 0 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (42)$$

Therefore, the point, G, does not move, which means that if the end of the arch were free, it would rotate about G. Since the end of the arch is fixed, it must develop a moment, given by Equation (43),

$$M = E \frac{\Sigma d(\Delta \phi)}{\sum \frac{\Delta L}{I}} = \frac{E I \alpha \Delta \tau}{t} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (43)$$

and, since,

$$I = \frac{t^3}{12}$$

$$M = \frac{E \alpha \Delta \tau}{12} t^2 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (44)$$

To find the stresses caused by the general and local variations of temperature is now easy. The former will bear, numerically, the same ratio to the stresses due to water load as H_D bears to H_p; that is, as $E \alpha D \tau$ is to $p r$. Algebraically, they will be subtracted from or added to those, according as the general temperature variation is a rise or fall. The stresses due to local temperature variation are equal for all arch sections and are given by,

$$\pm \frac{M}{Z} = \frac{\frac{E \alpha \Delta \tau}{12} t^2}{\frac{t^2}{6}} = \frac{E \alpha \Delta \tau}{2} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (45)$$

The negative sign is given to the stress that occurs at the face where the temperature is the greater, and the positive sign to the stress at the other face.

It is convenient to combine Equations (33) and (34) for stress into one expression giving the stress at the four critical points (see Equation (23)).

For brevity, let,

$$K_c = \frac{1}{K_0} \left[\frac{L - l}{L} - \frac{t}{6 r} \right] \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (46)$$

$$K_i = \frac{1}{K_0} \left[\frac{L - l}{L} + \frac{t}{6 r} \right] \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (47)$$

$$K_e' = \frac{1}{K_0}\left[\frac{l}{L} - \frac{r-h}{r}\left(1 - \frac{t}{6\,r}\right)\right] \quad \ldots\ldots\ldots\ldots (48)$$

$$K_i' = \frac{1}{K_0}\left[\frac{l}{L} - \frac{r-h}{r}\left(1 + \frac{t}{6\,r}\right)\right] \quad \ldots\ldots\ldots\ldots (49)$$

Then, the crown stresses are:

$$f_e = -\,p\left(\frac{r}{t} + K_e\right) + E\,\alpha\left(D\,\frac{t}{r}\,K_e - \frac{\Delta\,\tau}{2}\right) \ldots\ldots (50)$$

$$f_i = -\,p\left(\frac{r}{t} - K_i\right) - E\,\alpha\left(D\,\frac{t}{r}\,K_i - \frac{\Delta\,\tau}{2}\right) \ldots\ldots (51)$$

and the abutment stresses are:

$$f_e' = -\,p\left(\frac{r}{t} - K_e'\right) - E\,\alpha\left(D\,\frac{t}{r}\,K_e' + \frac{\Delta\,\tau}{2}\right) \ldots\ldots (52)$$

$$f_i' = -\,p\left(\frac{r}{t} + K_i'\right) + E\,\alpha\left(D\,\frac{t}{r}\,K_i' + \frac{\Delta\,\tau}{2}\right) \ldots\ldots (53)$$

The term, $\frac{\Delta\,\tau}{2}$, should be used as it stands only for relatively thin arches.

Table 2 is useful in showing at a glance the direction of the stress due to any given condition.

TABLE 2.—Direction of Stress for Any Given Condition.
(Compression —; Tension +.)

Section.	Water Load Stresses.		Variation of Temperature.			
			D.		$\Delta\,\tau$.	
	$p\,r$.	H_p.	Rise.	Fall.	Rise.	Fall.
Crown:						
Extrados	−	−	+	−	−	+
Intrados	−	+	−	+	+	−
Abutment:						
Extrados	−	+	−	+	−	+
Intrados	−	−	+	−	+	−

Crown Deflection Due to Water Load

Crown deflection takes place in the direction of the axis, Y, and may be regarded as the sum of that due to the uniform shortening of the arch under the stress, $\frac{p\,r}{t}$, and that due to the reaction, H_p, given by Equation (24).

The first deflection is obviously equal to,

$$\Delta\,h_k = \frac{p\,r}{E\,t}\,h \quad \ldots\ldots\ldots\ldots\ldots\ldots (54)$$

and the second, $\Delta\,h_H$, must now be determined.

Consider the half arch fixed at one abutment and loaded on the crown section by the reaction, H_p, passing through the elastic center of gravity, G,

of the whole arch (see Fig. 10). According to the theory of the ellipse of elasticity, the deflection, Δh_H, is given by the expression,

$$\Delta h_H = H_p\, g'\, d\, d_x \qquad (55)$$

in which,

$$g' = \frac{1}{2}\,\frac{L}{E\,I} = \frac{6\,L}{E\,t^3} \qquad (56)$$

The elastic center of gravity, G', of the half arch evidently falls on the x-x axis passing through G, and,

$$(\text{line } O\ G') = r\,\frac{r\sin\frac{\phi_0}{2}}{r\,\frac{\phi_0}{2}} = \frac{r\sin\frac{\phi_0}{2}}{\frac{\phi_0}{2}} \qquad (57)$$

Whence,

$$d = (\text{line } G\ G') = (\text{line } O\ G')\sin\frac{\phi_0}{2} = \frac{r\sin^2\frac{\phi_0}{2}}{\frac{\phi_0}{2}} \qquad (58)$$

Also,

$$d_x = (\text{line } G'\ G'') - (\text{line } Y\ Y') \qquad (59)$$

Now,

$$(\text{line } G'\ G'') = (\text{line } O\ G) = r\,\frac{\sin\phi_0}{\phi_0} \qquad (60)$$

and,

$$(\text{line } Y\ Y') = \frac{\int_0^{\phi_0} x\ y\ d\ L}{r\,\phi_0\,r\,\dfrac{\sin^2\frac{\phi_0}{2}}{\frac{\phi_0}{2}}} \qquad (61)$$

The expression to be integrated may be reduced as follows:

$$x\ y\ d\ L = r\ d\ \phi\sin\phi\,[(\text{line } m\ n) - (\text{line } e\ n)]$$

$$= r^3\left(1 - \frac{t^2}{6\,r^2}\right)\sin\phi\cos\phi\ d\ \phi \qquad (62)$$

Hence, from Equations (62) and 61):

$$Y\ Y' = r\left(1 - \frac{t^2}{6\,r^2}\right)\frac{\int_0^{\phi_0}\sin\phi\cos\phi\ d\ \phi}{2\sin^2\frac{\phi_0}{2}}$$

$$= \frac{r}{2}\left(1 - \frac{t^2}{6\,r^2}\right)(1 + \cos\phi_0) \qquad (63)$$

and,

$$d_x = r\,\frac{\sin\phi_0}{\phi_0} - \frac{r}{2}\left(1 - \frac{t^2}{6\,r^2}\right)(1 + \cos\phi_0) \qquad (64)$$

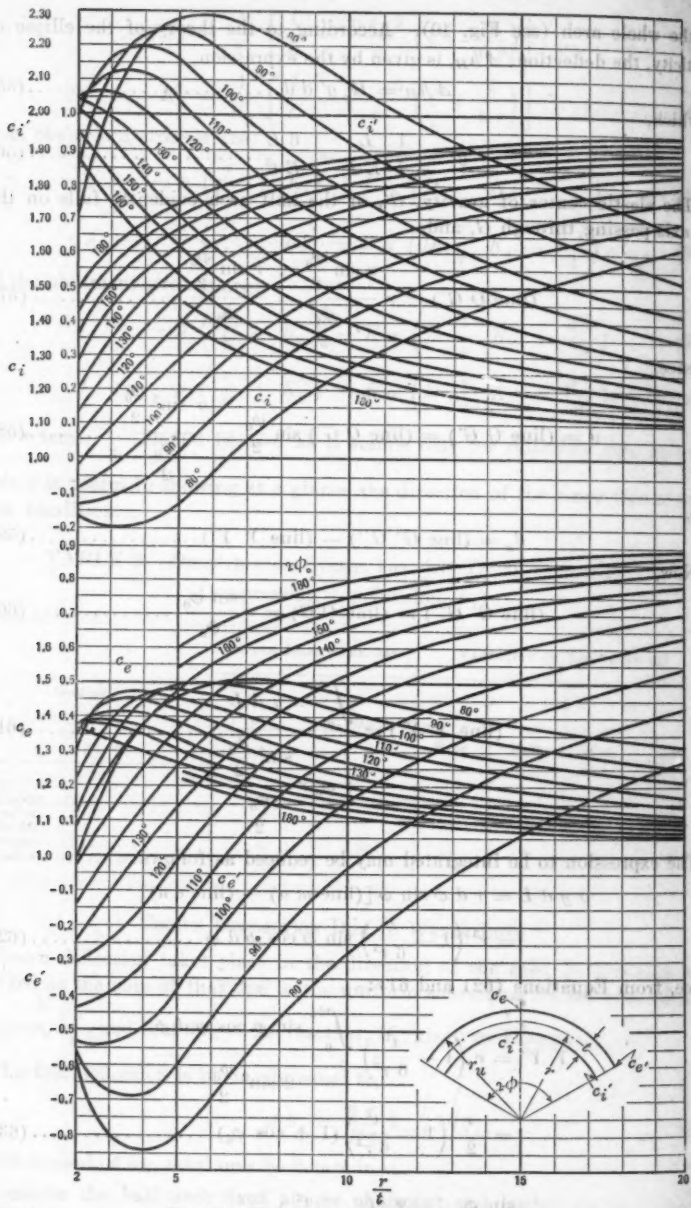

FIG. 11.—WATER LOAD STRESS COEFFICIENTS.

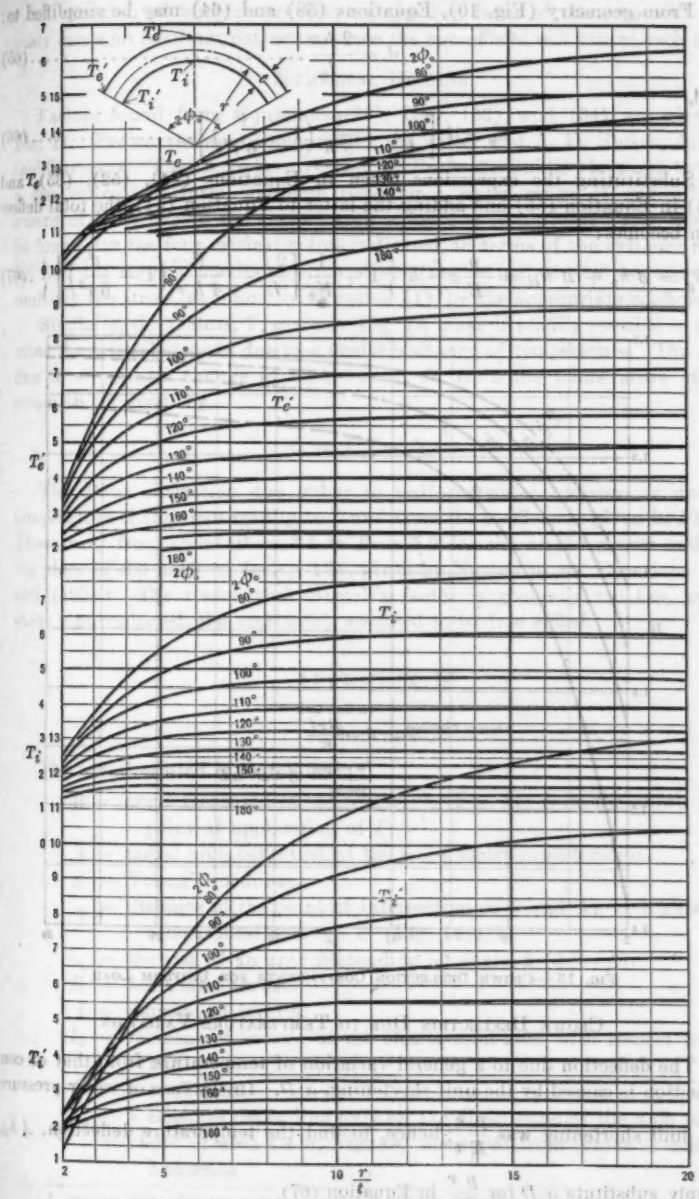

FIG. 12.—TEMPERATURE STRESS COEFFICIENTS.

From geometry (Fig. 10), Equations (58) and (64) may be simplified to:

$$d = \frac{2\,h\,r}{L} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (65)$$

and,

$$d_x = l\left[\frac{r}{L} - \frac{l}{8\,h}\left(1 - \frac{t^2}{6\,r^2}\right)\right] \quad \ldots\ldots\ldots\ldots\ldots\ldots (66)$$

Substituting the expressions given in Equations (24), (52), (65), and (66) in Equation (55) and adding the latter to Equation (54), the total deflection becomes:

$$\Delta h_p = \Delta h_k + \Delta h_H = \frac{p}{E}\,\frac{r}{t}\,h\left\{1 + \frac{1}{K_0}\left[\frac{2\,l}{L} - \frac{l^2}{4\,h\,r}\left(1 - \frac{t^2}{6\,r^2}\right)\right]\right\} \quad \ldots (67)$$

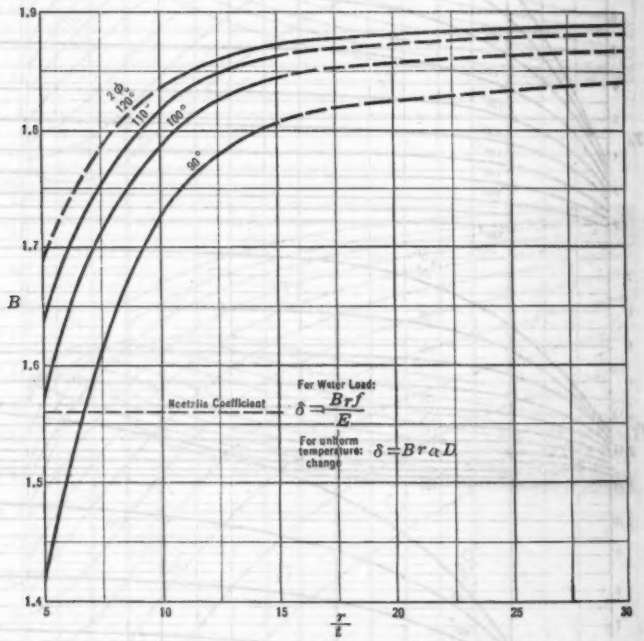

FIG. 13.—CROWN DEFLECTION COEFFICIENTS FOR UNIFORM LOAD.

CROWN DEFLECTION DUE TO TEMPERATURE VARIATION

The deflection due to a general variation of temperature from that of construction is caused by the unit shortening, $\alpha\,D$. In the case of water pressure, the unit shortening was $\frac{p}{E}\,\frac{r}{t}$; hence, to find the temperature deflection, Δh_D, simply substitute $\alpha\,D$ for $\frac{p}{E}\,\frac{r}{t}$ in Equation (67).

A variation of temperature between the two faces of the arch will obviously cause no resultant deflection, since the two effects will cancel each other.

Numerical Results

Factors found from Equations (28), (29), (33), and (34) are given in Fig. 11. The curves were computed by the writer and A. H. Stokes, Authorized Surveyor. They have been verified by Professor Guidi, who gives them in a less convenient form, although covering a wider range of angles.* By means of these curves the correct arch stress due to uniform water load can be found, for the four critical points indicated, in terms of the cylinder stress. To find the correct maximum arch stress (neglecting vertical beam action) multiply the stress, as found by Equation (1), by the appropriate coefficient.

Similarly, the factors, T, given in Fig. 12, make it readily possible to determine the critical stresses due to a uniform change of temperature. The stress due to a uniform change of temperature, D, from the temperature of construction, is given by,

$$f_\tau = T\,E\,\alpha\,D\,\frac{t}{r}\,.$$

The crown deflection due either to uniform water pressure or uniform temperature change can readily be found from the coefficients given in Fig. 13. These vary from about $B = 1.4$ to $B = 1.9$ for the angles shown and take the place of the uniform factor, 1.56, given by Noetzli in his Equations (13a) and (13b).† The reason that Noetzli's factor is generally too low, is that shear was neglected, the arch being assumed to be free ended.

APPENDIX II

Notation

The notation used is as follows:

β = angle measured at the center, O, (Fig. 14) from the crown to the point of application of P.
δ = radial arch deflection at the point under consideration.
E = Young's modulus.
η = distance of the point of intersection of P and the arch axis from a horizontal through C (Fig. 14(a)).
h_1 = the rise of an arch subtending an angle, $2\,(\phi_0 - \beta)$.
H = horizontal thrust.
I = moment of inertia.
I_x = moment of inertia of the elastic semi-arch with respect to Axis x-x.
K = a coefficient.
K_0 = a constant depending only on the dimensions of the arch section.
l = chord of arch ($A\,B$, Fig. 15) measured from center to center of haunches.

* "Statica delle dighe per laghi artificiali," Second Edition, 1926.
† *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 12.

L = length of mean arc.

L_1 = length of arc comprised between two symmetrical forces, P.

M = bending moment added to counteract the translation of the arch thrust, H, from the crown to the axis through the elastic center, G', of the half arch.

N = a coefficient = $\frac{\delta E I}{P r^3}$ (see Fig. 15).

p = pressure due to water load.

p_e = external pressure due to water load.

P = one of a pair of equal radial forces applied to the arch at equal angles, β, from the center line.

ρ = radial semi-axis of the ellipse of elasticity of the arch element.

ρ_1 = tangential semi-axis of the ellipse of elasticity of the arch element (Fig. 8).

q = distance from the line of action of the thrust, H, of the antipole, with respect to the elastic ellipse of the arch element, of the radius through the point under consideration (Line ND, Fig. 14(d)).

r = mean radius of arch.

r_u = radius of up-stream face.

s = Line SD, Fig. 14(d) = distance of antipole defined by q from the line of action of the force, P.

t = thickness of arch.

x-x = axis through the center of gravity of the mean arc and parallel to the chord (see Fig. 14).

X-X = axis parallel to x-x through the center of curvature.

y = distance from the axis, x-x, of the antipole of line of action of P with respect to the ellipse of elasticity of the arch element.

y_X = distance from the axis, X-X, of the antipole of line of action of P, with respect to the ellipse of elasticity of the arch element.

z = distance of the center of the arch element from the line of action of the force, P.

z_1 = distance of the center of the arch element from the radius through the point under consideration.

ζ = distance from the center line, of the point of intersection of P and the mean arc, or arch axis.

ϕ = angle measured at the center, O (Fig. 14), from the crown to the arch element.

ϕ_o = one-half the central angle measured at the center, O (Fig. 14).

ψ = angular distance from the center line of the point the deflection of which is to be found.

Deflection Produced by Non-Uniform Water Pressure

The hypothesis generally accepted in the design of arch dams is that the water pressure is uniform over any given horizontal arch slice, but this does not

correspond with fact. This conclusion, if it holds at all, can do so only for a dam in a site of rectangular section. In the vast majority of cases, with a V- and U-shaped site, the reverse is the case; that is, the pressure on the arch slice is greatest in the center. This is amply borne out by measurements on the Stevenson Experimental Dam.* The arch dam acts as a plate, and the pressure on any given arch slice increases from the abutments to the crown, which is the only place that it has full value due to the head of water.

It is of interest to examine the effect of such unequal load distribution on the stresses and deflections produced. Consider an arch slice of constant section and unit height, as shown in Fig. 14, and determine first the thrust deflection at the crown produced by two equal and symmetrical radial forces, P. The crown section will obviously not rotate nor be displaced in a direction normal to the radius through it. Hence, take the half arch, Fig. 14(a), solidly fixed at A and loaded by the force, P, and by the thrust, H, produced by the other half arch similarly loaded. By reason of symmetry the thrust, H, is normal to the radius, $O\,C$, and may be translated to pass through the elastic center, G', of the half arch by adding the moment, M, at G'.

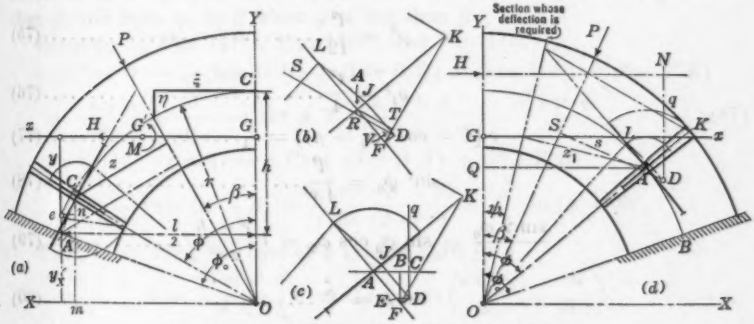

FIG. 14.—ARCH THEORY FOR CONCENTRATED LOADS.

It is then possible to write two equations expressing the fact that the rotation of the crown section is nil, as is also the deflection (in a direction normal to the Y-axis) of the elastic center of gravity, G', which is considered rigidly connected with the crown, C.

The first equation is,

$$-\frac{L}{2}M + P\int_{\beta}^{\phi_0} z\,dL = 0 \qquad (68)$$

Noting that $dL = r\,d\phi$, and integrating, this equation may be written:

$$M = 2\frac{P\,r^2}{L}[1 - \cos(\phi_0 - \beta)] = 2\,P\,r\,\frac{h_1}{L} \qquad (69)$$

The other equation is,

$$-H\,I_x + P\int_{\beta}^{\phi_0} z\,y\,dL = 0 \qquad (70)$$

* As reported in *Engineering News-Record*, November 18, 1926, p. 828.

Representing the elementary elastic weight simply by dL (to which it is proportional), the value of I_x is given by,

$$I_x = \frac{l\, r^2\, K_0}{4} \quad \ldots\ldots (71)$$

Furthermore,

$$z = r \sin (\phi - \beta) \quad \ldots\ldots (72)$$

and,

$$y = (\text{line } O\,G) - [(\text{line } m\,c) - (\text{line } n\,c)] = \frac{r \sin \phi_0}{\phi_0} - \left[\left(r + \frac{\rho^2}{r}\right) \cos \phi - \frac{{\rho_1}^2}{r \tan (\phi - \beta)} \sin \phi\right] \quad \ldots\ldots (73)$$

Hence,

$$\int_\beta^{\phi_0} z\, y\, dL = r^3 \left[\frac{\sin \phi_0}{\phi_0} \int_\beta^{\phi_0} \sin (\phi - \beta)\, d\phi - \left(1 + \frac{\rho^2}{r^2}\right) \int_\beta^{\phi_0} \sin (\phi - \beta) \cos \phi\, d\phi - \frac{{\rho_1}^2}{r^2} \int_\beta^{\phi_0} \sin \phi \cos (\phi - \beta)\, d\phi\right] \quad \ldots\ldots (74)$$

To simplify the integration, the following relationships are convenient:

$$\rho^2 = \frac{t^2}{12} \quad \ldots\ldots (75)$$

$${\rho_1}^2 = \frac{t^2}{4} \quad \ldots\ldots (76)$$

$$r\,[1 - \cos (\phi_0 - \beta)] = h_1 \quad \ldots\ldots (77)$$

$$\sin^2 \phi_0 = \frac{l^2}{4\, r^2} \quad \ldots\ldots (78)$$

$$\frac{\sin 2\,\phi_0}{2} = \sin \phi_0 \cos \phi_0 = l\, \frac{r - h}{2\, r^2} \quad \ldots\ldots (79)$$

$$\sin \beta = \frac{\zeta}{r} \quad \ldots\ldots (80)$$

$$\phi_0 - \beta = \frac{L - L_1}{2\, r} \quad \ldots\ldots (81)$$

and,

$$\cos \beta = \frac{r - \eta}{r} \quad \ldots\ldots (82)$$

Substituting Equations (75) to (82) in Equation (74):

$$\int_\beta^{\phi_0} z\, y\, dL = \frac{r^3}{2} \left[2\, \frac{l}{L}\, \frac{h_1}{r} - \left(1 - \frac{t^2}{6\, r^2}\right) \left(\frac{r - \eta}{r}\, \frac{l^2}{4\, r^2} - \frac{\zeta}{r}\, \frac{l}{2\, r}\, \frac{r - h}{r}\right) + \left(1 + \frac{t^2}{3\, r^2}\right) \frac{\zeta}{r}\, \frac{(L - L_1)}{2\, r}\right]$$

Hence,

$$H = \frac{2\, P}{K_0} \left[\frac{2\, h_1}{L} + \left(1 + \frac{t^2}{3\, r^2}\right) \frac{\zeta}{r}\, \frac{(L - L_1)}{2\, l} - \frac{1}{2} \left(1 - \frac{t^2}{6\, r^2}\right) \left(\frac{l}{2\, r}\, \frac{(r - \eta)}{r} - \frac{\zeta}{r}\, \frac{(r - h)}{r}\right)\right] \quad \ldots\ldots (83)$$

Taking Equations (69) and (83) for M and H, respectively, and substituting therein $p\,r\,d\,\beta$ for P (uniform water load), and integrating with respect to β:

$$M = p\,r\,(\text{line } G\,C) = p\,r^2\,\frac{L-l}{L} \quad \ldots\ldots\ldots\ldots\ldots\ldots (84)$$

$$H = p\,r\left(1 - \frac{t^2}{6\,K_0\,r^2}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (85)$$

M and H, thus obtained, are identical in effect with the value of H given in Equation (24), thus verifying the case of uniform water pressure as a special case of non-uniform pressure.

Deflection

The arch deflection at a point, the radius of which makes an angle, ψ (Fig. 14(d)), with the center line, is given by the expression,

$$\delta = \frac{H}{E\,I}\int_{\phi_0}^{\psi} z_1\,q\,d\,L - \frac{1}{E\,I}\int z\,s\,d\,L \quad \ldots\ldots\ldots\ldots\ldots (86)$$

in which, the integration of the second term is from ϕ_0 to ψ, when ψ is greater than β, and from ϕ_0 to β when ψ is less than β.

Referring to Fig. 14(b), Fig. 14(c), and Fig. 14(d):

$$q = (\text{line } O\,G) + (\text{line } G\,H) - (\text{line } O\,Q) + (\text{line } B\,F) - (\text{line } F\,E)$$

$$= \frac{r\sin\phi_0}{\phi_0} + \frac{M}{H} - r\cos\phi + \frac{{\rho_1}^2\sin\phi}{r\tan(\phi-\psi)} - \frac{\rho^2\cos\phi}{r} \quad \ldots\ldots\ldots\ldots (87)$$

$$s = (\text{line } S\,R) + (\text{line } A\,T) + (\text{line } V\,D)$$

$$= r\sin(\phi-\beta) + \frac{{\rho_1}^2\cos(\phi-\beta)}{r\tan(\phi-\psi)} + \frac{\rho^2}{r}\sin(\phi-\beta)$$

First, integrating the case of ψ greater than β:

$$\delta = -\frac{H\,r^3}{4\,E\,I}\left[4\,\text{vers}(\phi_0-\psi)\left(\frac{\sin\phi_0}{\phi_0} + \frac{M}{H\,r}\right) + \left(1 - \frac{t^2}{6\,r^2}\right)(\cos(2\,\phi_0-\psi) - \cos\psi) + 2\left(1 + \frac{t^2}{3\,r^2}\right)(\phi_0-\psi)\sin\phi\right]$$

$$- \frac{r^3}{4\,E\,I}\left[\left(1 - \frac{t^2}{6\,r^2}\right)(\sin(2\,\phi_0-\beta-\psi) - \sin(\psi-\beta)) - 2\left(1 + \frac{t^2}{3\,r^2}\right)(\phi_0-\psi)\cos(\psi-\beta)\right] \quad \ldots\ldots\ldots\ldots (88)$$

For the case when ψ is less than β, the quantity $(\psi-\beta)$ in the second term of Equation (88) becomes $(\beta-\psi)$ and the quantity, $(\phi_0-\psi)$, becomes $(\phi_0-\beta)$.

The writer has computed values of $\frac{H}{P}$, $\frac{M}{H\,r}$, and δ, for the subtended angles, 90°, 100°, 110°, and 120°, and for various thicknesses of arch (see Fig. 15). For each subtended angle, the deflection curves have been plotted for three different values of β, namely, $\beta = 0$, 15, and 35 degrees. The coefficients

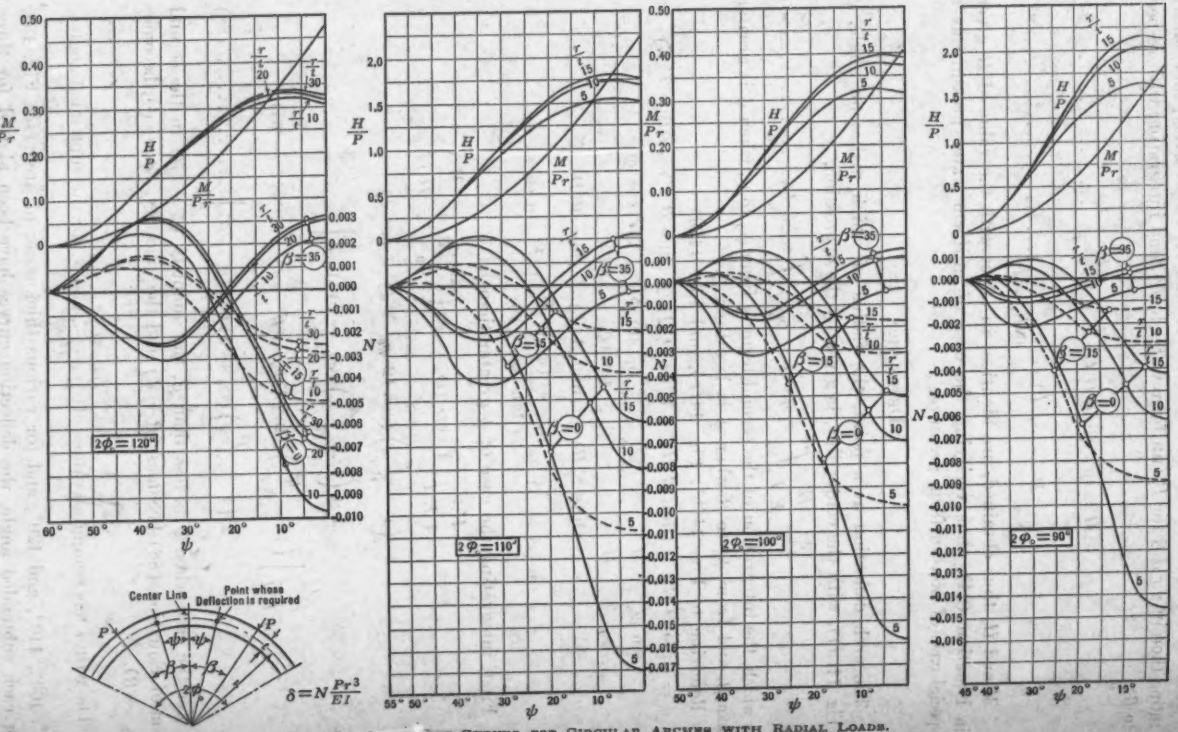

FIG. 15.—INFLUENCE CURVES FOR CIRCULAR ARCHES WITH RADIAL LOADS.

shown on the diagrams are for the imaginary case when P, r, E, and I are each unity, and for any given case the actual deflection is obtained by multiplying the coefficient by $\frac{P r^3}{E I} = \frac{12 P}{E} \left(\frac{r}{t}\right)^3$.

The values of $2 \phi_0$ and $\frac{r}{t}$ cover most cases that occur in practice, and deflection coefficients for intermediate values may readily be found by drawing subsidiary interpolation curves.

From the method of deriving the curves (Fig. 15), and by reason of Maxwell's theorem of reciprocity, it will be seen that they are influence curves for the various quantities they represent, and may be used to find these quantities for any conditions of loading; that is, either concentrated loads, uniformly distributed loads, or non-uniformly distributed loads. The last case is the one which is of immediate interest.

Fig. 16 will serve to show the method of finding the moment, thrust, and deflection in a particular case. The data assumed are as follows:

$r = 100$ ft.

$t = 6.67$ ft.

$\frac{r}{t} = 15$

$2 \phi_0 = 90°$

Water load = 3 000 lb. per sq. ft. at crown, and varying as shown in the load diagram.

$$p = p_c \frac{r_u}{r} = 3\,000 \times \frac{103.33}{100} = 3\,100 \text{ lb. per sq. ft.}$$

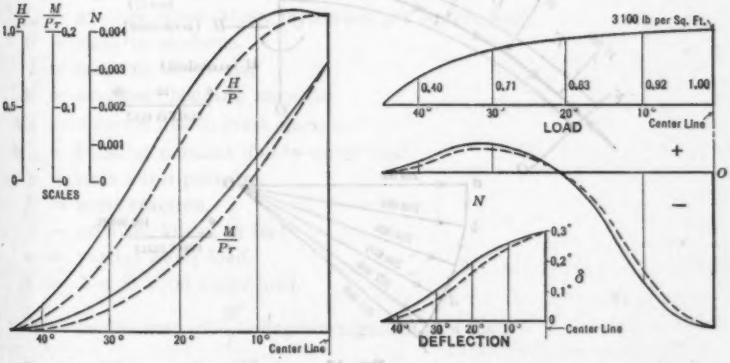

FIG. 16.—METHOD OF FINDING MOMENT, THRUST, AND DEFLECTION IN A GIVEN CASE.

Now, P can be regarded as the concentrated load pressing on the portion of arch subtended by 1 degree. Hence,

$$P = 3\,100 \times \frac{\pi}{180} \times 100 = 5\,410 \text{ lb. at the crown}$$

If P were to remain constant in magnitude, but were to move to various points of the arch, the value of $\frac{M}{H\,r}$, H, and N (for crown deflection), would be read off from the ordinates of the curves shown in full lines corresponding to the position of P. Since P reduces as it moves away from the crown, these ordinates must be reduced in the same manner. Thus is derived the dotted curves, the ordinate at any point of which would give the value of the quantity required for a concentrated radial load acting at that point, and equal in magnitude to the actual load pressing on an arc of 1° at that point. Regarding the actual load as a series of concentrated loads it is seen that the value of the quantity required, under the actual conditions of loading, is obtained by finding the total area under the dotted curves.

In the case shown the areas are: For $\frac{H}{P}$, 48 degree-lb. units; for $\frac{M}{P\,r}$, 5.02 degree-ft-lb. units; and for N, 0.0376 degree-ft. units. Hence,

$$H = 5\,410 \times 48 = 259\,000 \text{ lb.}$$

$$M = 5\,410 \times 100 \times 5.02 = 2\,715\,000 \text{ ft-lb.}$$

and, taking,

$$E = 288\,000\,000 \text{ lb. per sq. ft.}$$

$$\delta = \frac{12 \times 5\,410 \times 15^3 \times 0.0376 \times 12}{288 \times 10^6} = 0.342 \text{ in.}$$

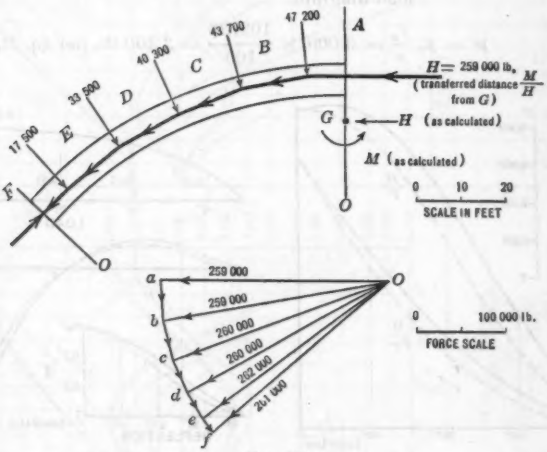

FIG. 17.

The deflection influence lines for $\beta = 15°$ and $35°$ have not been shown, but the deflections at these points can be found in exactly the same manner (Fig. 16). The developed deflection curve for the arch would be as shown in dotted lines, the corresponding curve for a uniform load of 3 000 lb. per sq. ft.

being shown in full lines. The importance of taking into account the distribution of arch load will be clear from this example.

To determine the stresses in the arch, it is necessary to sketch the conditions of loading as shown in Fig. 17, in which the water load has been replaced by concentrated loads acting on each 9° of arc. It is to be noted that five forces only were taken for the sake of simplicity. In practice, further sub-division is necessary. Now, the moment, M, was introduced to neutralize the moment caused by transferring the line of action of the thrust, H, to pass through the elastic center of gravity, G. The distance from G to the actual line of action of H is, therefore, $\frac{M}{H} = 10.47$ ft. This locates H, and the determination of the resultant force acting at any point can then be effected by the usual procedure. The value and position of the thrust at any point being known, the stresses can be found by the trapezium rule.

APPENDIX III

Notation

The following notation has been used:

a = distance between sections assumed for the purpose of analysis.
d = lever arms of partial water loads (Fig. 18).
d_R = moment arm to crest reaction.
d_z = moment arm to areas on $\frac{M}{I}$-diagram.
δ = deflection.
E = Young's modulus.
f_s = average shear stress, in pounds per square foot.
G = rigidity modulus.
I = moment of inertia.
M = resultant bending moment.
M_R = moment due to crest reaction.
M_w = bending moment due to water load.
p = unit water pressure.
R = crest reaction.
t = arch thickness, in feet.
w = partial water load.
$W = \Sigma w$ = total water load.
z = partial area of $\frac{M}{I}$-diagram regarded as a load.
Z = total area of the $\frac{M}{I}$-diagram regarded as a load $= \Sigma z$.

Arithmetical Computation of Beam Deflections

The following arithmetical method of finding the beam deflection curves is based on the well-known fact that the bending moment diagram of a beam

or cantilever loaded with its $\frac{M}{I}$-diagram, will represent, to a suitable scale, the deflection curve. This fact is derived from the elementary theory of beams, which may be summarized in the five expressions:

$$\begin{aligned} E\,I\,y'''' &= \text{load for unit length;} \\ E\,I\,y''' &= -\text{ shear;} \\ E\,I\,y'' &= \text{moment;} \\ E\,I\,y' &\text{ gives the slope; and,} \\ E\,I\,y &\text{ gives the deflection.} \end{aligned}$$

It is obvious from these expressions that the process of repeated integration by which the bending moment is derived from the load diagram will also make it possible to find the deflection from the $\frac{M}{I}$-diagram, using an appropriate scale. This is the basis of Mohr's graphical method. The writer considers that a tabular arithmetical method of doing the same thing is preferable because it eliminates inaccuracies due to errors in drawing and the work can be done very quickly by an intelligent computer, who need not have any knowledge of the theory on which the work depends.

Table 3 gives, in a convenient form, values of I, load, bending moment, etc. Consider the irregular beam, $A\ B\ C\ D$, of unit width (Fig. 18(b)) with a load diagram as shown by $E\,F\,G\,H\,I$, and the reaction, R (Fig. 18(a)). The beam is divided by any convenient number of equi-distant Sections 1, 2, 3, etc., spaced a units apart, and the points of application of the partial loads between the sections are determined, either by the simple geometrical construction or by the simple formula for finding the center of gravity of a trapezium. When the distance between sections is small compared with the total length of the beam, as is generally the case, it may be sufficiently accurate to assume the partial loads to be applied midway between the sections, except for the uppermost section, where the distance is $\frac{a}{3}$ from Section 1.

In tabulating the moments:

$$\begin{aligned} M_1 &= w_1 \times d_1 \\ M_2 &= w_1\,(a + d_1) + w_2\,d_2 \\ &= M_1 + w_1\,a + w_2\,d_2 \\ M_3 &= M_2 + (w_1 + w_2)\,a + w_3\,d_3 \end{aligned}$$

Hence, at any section simply add the progressive moment up to the preceding section, the progressive load multiplied by the constant distance, a, and the moment due to the partial load between the section considered and the preceding section. By making a equal to 20 ft. and assuming that the partial loads act midway between the sections, the arithmetic is reduced to the greatest simplicity.

The bending moment caused by the crest reaction R, is the product of the force and its distance from the section considered. This bending moment is thus simply computed for the various sections and deducted from the "water

TABLE 3.—BENDING MOMENTS AND DEFLECTIONS.

Section.	t, in feet.	I.	p, in pounds.	p (average), in pounds per foot.	a, in feet.	w, in pounds.	$W = \Sigma w$.	d, in feet.	M_w, in foot-pounds (added downward).	Remarks.
0	10	83.4	0	800	20	6 000	6 000	6.67	40 000	$= d\,w$
								20.00	120 000	$= d\,W$
1	12	144.0	600	1 050	20	21 000	27 000	10	210 000	$= d\,w$
								20	370 000	= sub-total
									540 000	$= d\,W$
2	14	228.0	1 500	2 625	20	52 500	79 500	10	525 000	$= d\,w$
3	20	667.0	3 750	.....	....	.....	.....	.....	1 435 000	= Total

d_R, in feet.	$M_R = R\,d_R$, in foot-pounds.	$M = M_w + M_R$, in foot-pounds.	$\frac{M}{I}$	z.	$Z = \Sigma z$ (added upward).	d_z, in feet.	$E\,\delta$ (added upward), in foot units.	Remarks.	δ, in inches.
							1 710 200	= Total.	0.071
.....	.....	.....	.....	−920	37 260	....	9 200	$= z\,d_z$	
						10	763 600	$= Z\,d_z$	
13.3	— 53 300	— 13 300	−92	9 480	38 180	20	955 800	= sub-total.	0.040
						10	94 800	$= z\,d_z$	
33.3	−133 300	+226 700	1 040	28 700	28 700	20	574 000	$= Z\,d_z$	0.012
						10	287 000	$= z\,d_z$	
53.3	−213 300	+1 221 700	1 880	.....	.....	....	.....	.....	.....

moment", M_w, giving the resultant moment, from which $\frac{M}{EI}$ is found. These values are then regarded as the ordinates of the new load diagram, and the same process is repeated, in this case starting at the bottom and working upward.

Table 3 shows the method of computing points on the deflection curve for the beam dimensions and loads shown in Fig. 18, only three sections being taken for the sake of simplicity. It will be seen that in this method allowance can very easily be made for any non-uniformity in dimensions or loads. Thus, if the ratio of thickness to radius of curvature is large, the value of I given by Equation (44) may be corrected by the use of the curve given in Fig. 3.

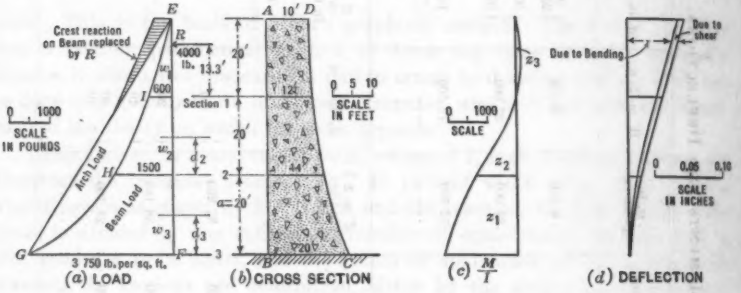

FIG. 18.—BEAM LOADS AND DEFLECTIONS.

Shear deflection is computed separately where necessary, by taking in turn the portions of beam between each pair of sections, and considering each portion subjected to a uniform shear stress equal to the average net load

TABLE 4.—SHEAR DEFLECTION.

Section.	t, in feet.	Load, in pounds.	Stress, in pounds per square foot.	Mean stress, in pounds per square foot.	Partial deflection, in inches.	Total deflection, in inches.
3	20	75 500	3 780	.....	.....	.....
2	14	23 000	1 640	2 710	0.006	0.006
1	12	2 000	170	905	0.002	0.008
0	10	0	0	85	0.000	0.008

divided by the average thickness. The method of computing is obvious from Table 4. The rigidity modulus, G, is assumed to be 115 000 000 lb. per sq. ft., and the deflection per 20 ft. $= \frac{20 \times 12}{115 \times 10^6} f_s$ in.

APPENDIX IV

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DISCUSSION

WILLIAM CAIN,* M. AM. SOC. C. E. (by letter).—The author has advanced the solution of the arch dam very materially by his analysis and the construction of the many diagrams as an aid in effecting the extended numerical computations required. The solution is a practical one for the dam of uniform thickness when the structure is not cracked or subjected anywhere to greater tension than the concrete can stand. However, if there is sufficient tension at certain sections to cause cracks where a load is suddenly applied, it may be best to consider these cracks in order to be on the safe side; although for the very slow application of the water load, as the author contends, the "plastic flow" of concrete will modify results and perhaps at some sections prevent "the tensile stresses from reaching the point indicated by theory." This is a consoling theory, but its realization was not much in evidence at the Stevenson Creek Experimental Dam where a large number of cracks formed. The theory must fit the facts.

As nearly all dams are not fixed at the base, in the sense of supplying sufficient tensile resistance there, cracks will occur at the base and a theory which ignores them cannot give correct results.

In the paper entitled "Analysis of Arch Dams by the Trial Load Method,"† by C. H. Howell, M. Am. Soc. C. E., and the late A. C. Jaquith, Esq., the parts of the sections of both arches and cantilevers in tension are neglected, and only the part of the section in compression is regarded as effective in computing moments. This, of course, is going to one extreme, for concrete can resist a tension of 100 to 200 lb. per sq. in., and the possible modification due to plastic flow is neglected, but the hypothesis is on the safe side. In fact, considering other influences that are usually neglected, such as shrinkage, variations in E, water-soaking, and the elastic yielding of the foundation and abutments, it seems best to take the extreme view and consider as effective, for resistance to moments, only the part of a section in compression, particularly where the supposed crack covers a large part of the cross-section.

The author's analysis makes use of the ellipse of elasticity and thus abbreviates certain developments, and especial thanks are due him for the many labor-saving diagrams presented. For a dam that remains intact under the loads and temperature changes, the solution is complete, and even when cracks may be expected, the solution is valuable in a preliminary investigation. As the author affirms, the application of the method to many arch dams, leads to the conclusion that the normal load on a horizontal arch is not uniform.

One phase of the author's method is not clear. It is well known that there is one point on any assumed vertical beam (or cantilever),‡ considerably below the crest, where all the water load is carried by the horizontal arch and

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† See p. 1191.

‡ Point K, Fig. 15, in the writer's discussion of the paper entitled, "Gravity and Arch Action in Curved Dams," by Fred A. Noetzli, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 74.

none by the vertical beam. In Fig. 5 or Fig. 18 no such point seems in evidence.

The writer has computed by the work method,* the horizontal thrust, P_0, and the moment, M_0, at the crown for the circular arch of uniform thickness, t, and the central angle, $2\,\phi_0$, as well as the radial deflection anywhere for the two symmetrical radial loads, P, each making the angle, β, with the crown section (Fig. 15), but the resulting formula for deflection was found to be more cumbersome than the one derived by the author. The formula for the horizontal thrust, P_0, is,

$$P_0\,D_n = [(\phi_0 - \beta)\sin\beta - \sin\phi_0 \sin(\phi_0 - \beta)]\left(1 + \frac{k^2}{r_n{}^2}\right) + 2.88\,\frac{k^2}{r^2}[(\phi_0 - \beta)\sin\beta + \sin\phi_0 \sin(\phi_0 - \beta)] + 2\,\frac{\sin\phi_0}{\phi_0}[1 - \cos(\phi_0 - \beta)]$$

in which,

$k^2 = \frac{1}{12}\,t^2$; r_n = radius of neutral axis;

$\frac{k^2}{r_n{}^2} = \frac{1}{12}\left(\log_e \frac{r_e}{r_i}\right)^2$;

r_e = radius of extrados;

r_i = radius of intrados; and,

$$D_n = (\phi_0 + \tfrac{1}{2}\sin 2\,\phi_0)\left(1 + \frac{k^2}{r_n{}^2}\right) - \frac{1 - \cos 2\,\phi_0}{\phi_0} + 2.88\,\frac{k^2}{r_n{}^2}(\phi_0 - \tfrac{1}{2}\sin 2\,\phi_0)$$

The thrust, P_0, is assumed to be exerted at the center of the crown joint and the corresponding moment there, counter-clockwise moments being positive, is:

$$M_0 = P\,r_n\,\frac{1 - \cos(\phi_0 - \beta)}{\phi_0} - P_0\,r_n\left(1 - \frac{\sin\phi_0}{\phi_0}\right)$$

The author takes the horizontal thrust, H, as acting at the elastic center of the arch with a corresponding moment,

$$M = P\,r\,\frac{1 - \cos(\phi_0 - \beta)}{\phi_0}$$

This shows the relation between M and M_0. Presumably, when the mean radius, r, is substituted for r_n, P_0 should equal H.

As a check on the formulas for P_0 and M_0, $p_e\,r_e\,d\,\beta$ (p_e being the unit load on the extrados), was substituted for P and the result integrated between the limits, $\beta = 0$ and $\beta = \phi_0$, to give P_0 and M_0 for the full normal load, and the results were found to agree exactly with previously derived formulas.†

* The derivation being similar to that used in *Transactions*, Am. Soc. C. E., Vol. 90 (June, 1927), pp. 529, 541.

† *Transactions*, Am. Soc. C. E., Vol. 90 (June, 1927), Equations (103) and (105), p. 531.

In the formula for P_0, the term having the factor, 2.88, shows the influence of shear; and since there is some uncertainty as to its numerical value the figure, 3, can be substituted for 2.88, to agree with the author's assumption. The hope of the designer is that some method of shortening the work materially by means of curves will be devised. In that direction, the author has gone a long way, and he will receive the thanks of the profession.

B. F. JAKOBSEN,* M. AM. SOC. C. E. (by letter).—In Appendix I the author gives the derivation of formulas for stresses, adapted from the work of Professor Guidi. These formulas apply for thin arches only, because of the assumption that the elastic center of gravity coincides with the center of gravity of the geometrical axis (see Fig. 7 and Equation (15)).

The distance, $O\,G$, is the writer's A', and this is:†

$$A' = r_n \frac{\sin \phi_0}{\phi_0} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (89)$$

in which, r_n is the radius to the neutral axis and ϕ_0 is one-half the central angle. When r_m is the mean radius, or the radius of the geometrical center line, then:‡

$$r_n = r_m - c \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (90)$$

and,

$$c = \frac{k}{1 + k} r_m$$

and for a rectangular section,

$$k = -1 + \frac{r_m}{t} \log_e \left(\frac{r_e}{r_i}\right) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (91)$$

in which, r_e and r_i are the radii of the extrados and intrados, respectively. The assumption, made by the author, that $c = 0$, is true, or approximately true, only for thin arches, as was shown by the writer and also by William Cain, M. Am. Soc. C. E.§ The author also assumes that the trapezium law (see Equation (25)) is correct; this also is approximately true only for thin arches,‖ as previously shown by the writer.

The derivation of stresses, as given by the author in Appendix I, is based on the same assumptions as were made by Professor Cain,¶ and the results, therefore, should be identical. As a check, assume an arch for which $\frac{t}{r} = 0.2$ and one-half the central angle, ϕ_0, = 45 degrees. From Equation (22) is found, $K_0 = 1.0267$ and from Equation (34), $f_1' = -11.98\ p$, whereas Professor Cain's formulas give $12.295\ p$, a difference of less than 3%, which probably is due to inaccuracies in the calculations. To the writer's way of thinking, Professor Cain's derivation is simpler and clearer, and the same holds

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† "Stresses in Thick Arches of Dams," *Transactions*, Am. Soc. C. E., Vol. 90 (June, 1927), Equation (18), p. 484; see, also, Professor Cain's discussion, Equation (105), p. 531.

‡ *Loc. cit.*, Equation (21), p. 485.

§ *Transactions*, Am. Soc. C. E., Vol. 90 (June, 1927), Equations (18) and (105), pp. 484 and 531.

‖ *Loc. cit.*, Fig. 52, p. 581.

¶ "The Circular Arch Under Normal Loads," *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 233.

for the writer's own derivation, as given in the paper referred to. That may be only a personal preference and is at any rate immaterial, the essential point being that the author's equations are true only for thin arches, such as $\frac{t}{r}$ less than 0.3. The same remark applies to his calculation of stresses due to temperature variations and to his calculations for deflections.

In investigating the deflection produced by non-uniform water pressure, he assumes that the load is symmetrical and that the two semi-arches are symmetrical also. The assumption that the load is constant for any arch is undoubtedly always incorrect, and so is the assumption of symmetry (see, for example, Fig. 1(*a*) and Fig. 1(*c*)). The writer's experience with the design of arch dams leads him to believe that it may not be worth while to attempt to take account of the non-uniform load on the arch, unless the question of symmetry is likewise taken into consideration. After all, the main purpose is to design a safe dam and, therefore, to make such assumptions as will lead to a stress determination which is first of all safe and, in addition, a fairly close approximation to the actual stresses. It is in this respect that the cylinder formula is lacking, because the stresses found are too small and sometimes much too small.

The author seems to have assumed throughout his paper that the arch load at the abutment is zero (see Fig. 5(*d*) and Fig. 5(*j*), and Fig. 16). This assumption is not correct, as the writer has pointed out in discussing another paper.* The author also assumes that the foundation is practically unyielding. The writer raised this question some years ago† and Fredrik Vogt, Assoc. M. Am. Soc. C. E., has dealt with it recently and has shown that both the cantilever deflections and the arch deflections are materially increased by the yielding of the foundation‡ and that the division of load between the cantilevers and the arches is influenced by the yielding of the foundation to a considerable degree. The measurements on the Stevenson Creek Test Dam show yielding of the foundation, which checks closely with the formulas of Dr. Vogt.§ The author also neglects shrinkage, but as the writer has recently dealt with this quite extensively,‖ there seems to be no necessity for repeating it. The report on the Stevenson Creek Test Dam also shows that shrinkage produced a very considerable cracking of the dam, and it is not clear to the writer that this can be neglected as being inconsiderable in calculating the stresses.

The author suggests that arch dam design may be improved by varying the radius of the horizontal arches, in order to adapt them better to the non-uniform arch load. Stucky¶ assumes that the center line of the arch coincides with the funicular polygon and that, therefore, generally the center line is not an arc of a circle. He does not, however, investigate the problem, but

* See p. 1226.

† *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 102.

‡ *Loc. cit.*, Vol. 90 (June, 1927), p. 554.

§ *Proceedings*, Am. Soc. C. E., May, 1928, Pt. 3, Report on Arch Dam Investigation p. 127, and Figs. 81 and 82, p. 128.

‖ *Transactions*, Am. Soc. C. E., Vol. 92 (1928), p. 832.

¶ "Etude sur les barrages arqués," Lausanne, 1922, "Calcul des arcs," p. 13.

states that in most cases the center line will vary so little from a true circle, that it may be assumed to be circular. Some years ago, after reading the paper by C. S. Whitney, M. Am. Soc. C. E., entitled "Design of Symmetrical Concrete Arches",* the writer made several attempts to determine the most economical shape of the center line. He came to the conclusion that due to the many uncertainties, such as shrinkage, temperature variations, yielding of the foundation, etc., the problem is too complicated and moreover that the shape of the most economical center line will vary as the volume of concrete, due to temperature variations, shrinkage, time effect, etc., so that little if anything can be accomplished with any degree of certainty.

The possibility of improving arch dams lies in the direction of a rational grouting of the contraction joints. In this way the dam may be made into a monolithic structure, which will have compressive arch stresses in places when the reservoir is empty; but it is not of vital importance to know whether the stress is 500 lb. per sq. in., or 10% more or less. It is important, however, to know that tension does not exist, since concrete cannot be depended on to resist tension. Dr. Vogt has introduced a system of grouting, which permits of varying the grouting pressure so that, for instance, a low grouting pressure is applied at the intrados at the abutment, where the compression would otherwise be high, and a high grouting pressure can be applied at the extrados of the abutment where otherwise a considerable tension may develop. It seems to the writer that such a system of grouting would be of considerable value in perfecting rational design and in the construction of arch dams.

The author correctly states that the cantilevers must be considered as having vertical radial limiting planes and not vertical parallel planes, as is generally assumed. This is of especial importance in gravity dam design, as the writer has shown.†

The author's Fig. 2 should be of assistance in determining the division of the water load between cantilevers and arches. If an approximation can be obtained on which to base the first set of calculations, these may be materially shortened, and even if absolute agreement is not reached, the result may be close enough for a stress determination.

PAUL BAUMAN,‡ M. AM. SOC. C. E. (by letter).—This paper is of particular value as it represents the latest results of arch dam research from all over the world. If properly applied, this method of analysis will make possible the determination of maximum stresses with sufficient accuracy to satisfy the most conservative requirements of engineering design. These requirements pertain to engineering practice and must be kept apart from purely scientific requirements as long as dam designing is done by engineers and not by physicists.

The elastic behavior of an isotropic substance is fully described by the modulus of elasticity, E, and the coefficient of transverse contraction, m (Poisson's ratio). The stresses in such a substance may be determined by the commonly known formulas. For anisotropic substances obeying Hooke's law, however, E and m must be replaced by twenty-one different constants

* *Transactions*, Am. Soc. C. E., Vol. 88 (1925), Article IV, p. 966.
† "Notes on Arched Gravity Dams," *Transactions*, Am. Soc. C. E., Vol. 92 (1928), p. 766.
‡ Designing Engr., Quinton, Code & Hill, Los Angeles, Calif.

which will quickly make the derivation of respective formulas impossible for practical use. What it would mean to make a theoretically exact analysis of a body of concrete, such as an arch dam, may be realized if consideration is given to the fact that concrete is not only anisotropic, but that it misbehaves with regard to Hooke's law. The dam designer, therefore, must be thankful for simple formulas, provided they are on the side of economy and safety. The stresses thereby determined in various points of the structure must then be considered as average stresses of a certain zone around these points.

The tests on the Stevenson Creek Dam have shown that the elastic theory holds good for relatively thin arches, which is a corroboration of many tests on bridge arches previously made in the United States and abroad; but the Stevenson Creek Dam is not representative of a high arch dam, and neither is the stress distribution as shown by the tests. The writer believes that the flow or plasticity of concrete in a high, massive arch dam of considerable volume alone will materially affect the load and the stress distribution. This will be a favorable influence due to its equalizing tendency.

The assumption of unyielding arch abutments is justifiable if the rock that forms the abutments is solid and is at least as strong as the concrete in the arch. It would not be in accordance with good engineering judgment to build an arch dam at a site not conforming to this condition. Even a gravity dam with sufficient curvature to permit arch action would be an unwise selection because the arch pressure of the top part is at times likely to be much in excess of the bottom pressure of the cantilever of maximum height. Consequently, a displacement of the arch abutments—with its well-known destructive effect on a hingeless non-reinforced arch—is quite possible.

The designer is often asked to prepare a preliminary design of an arch dam for the purpose of a cost estimate within such a short time that an accurate analysis is out of the question. It is then necessary to use a short cut which leads to both a safe and an economical result. The time element enters into the proposition beginning with the design! The number of sections for which the deflections are analyzed must then be reduced to a minimum, possibly to sections through the center and the quarter-points, or even to just one section through the line of maximum height. In the case of extremely short notice the writer has found that, in designing a dam, as much as 300 ft. in height, for arch action only, allowing an initial stress of 400 lb. per sq. in. at the crown and 365 lb. per sq. in. at the springing line, the volume of concrete will compare favorably with that obtained through accurate design for dam sites of ordinary, that is, parabolic, shape. The tensile stresses in the vertical beam which may be disclosed by an accurate design should never cause much concern inasmuch as the reinforcing steel required to take care of them will seldom exceed 1% of the cost. An investigation of the Sawpit Canyon Dam of the Los Angeles County (Calif.) Flood Control System, for example, has shown that the tensile stresses in the vertical beam due to simultaneous influence of full water load, shrinkage, drop in temperature, and elastic yield of abutments, could have been safely provided for with $1 700 worth of reinforcing steel (less than 0.3% of the cost). Since the introduction of reinforced concrete, the fear of tensile stresses is unwarranted.

The writer has often used the ellipse of elasticity both in bridge and dam design and, therefore, is pleased to see it used by the author. While the equations for elastic thrust and deflections due to uniform load are not simpler than those obtained by the work method, the elastic ellipse affords an excellent analysis of thrusts and deflections due to non-uniform and non-symmetric loads. In the case of a circular arch under uniform load (see Fig. 19), assume that the arch encastré at the right end and free at the left, is perfectly rigid. Obviously, then, the only condition for equilibrium is that the tangential reactions to the center line of the arch at each end, if combined with the resultant of the load, must form a closed triangle. Starting from the free end and letting one arch element after the other become elastic, the theory of the ellipse of elasticity shows that each element, under the influence of the resultant, N, of all the forces acting at a distance, S, from its center of gravity, will undergo an angular displacement, $d(\Delta\delta) = N S\, dg$. This angular displacement or rotation takes place about the antipole of the force, N, with respect to the ellipse of elasticity of the element. These elemental rotations may be considered as weights acting at the antipole of each element. If, therefore, the whole arch has become elastic, its angular displacement must be the sum of all the elemental displacements and must take place about the center of gravity of the elemental rotations acting as weights.

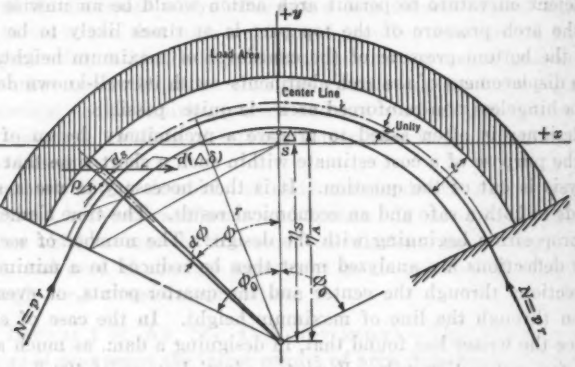

FIG. 19.—CIRCULAR ARCH, UNIFORMLY LOADED.

The supplementary force, ΔH, necessary to establish equilibrium of the elastic arch, may be found from the conditional equation:

$$\Delta H f \int_0^{\phi_0} d g = \int_0^{\phi_0} d(\Delta\delta)$$

or,

$$\Delta H = \frac{\int_0^{\phi_0} N S\, d g}{f \int_0^{\phi_0} d g} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (92)$$

in which, f is the normal distance of ΔH from the elastic center of the whole arch.

For a circular arch under uniform load the line of pressure coincides with the center line and $S = 0$. To make Equation (92) applicable it is necessary to move the line of pressure concentrically up or down by a constant distance, S, which, if made unity, at once disappears from the equation.

Therefore,

$$\Delta H = \frac{N \int_0^{\phi_0} d\,g}{f \int_0^{\phi_0} d\,g} = \frac{N}{f} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (93)$$

Using the author's notation:

$$N = p\,r$$

$$f = \frac{i_x^2}{a} = \frac{I_x}{a \int_0^{\phi_0} d\,g}$$

$$a = \eta_A - \eta_S \text{ (Fig. 19)}$$

$$\eta_S = \frac{r \sin \phi_0}{\phi_0}$$

$$d\,(\Delta\,\delta) = r\,p \times 1\,d\,g$$

$$\eta_A = \frac{\int_0^{\phi_0} \left(r + \frac{t^2}{12}\right) \cos \phi\, d\,(\Delta\,\delta)}{\int_0^{\phi_0} d\,(\Delta\,\delta)} = \frac{\left(r + \frac{t^2}{12}\right) r^2\,p}{E\,I} \frac{\int_0^{\phi_0} \cos \phi\, d\,\phi}{\frac{r^2\,p}{E\,I} \int_0^{\phi_0} d\,\phi}$$

$$= \frac{\left(r + \frac{t^2}{12}\right) \sin \phi_0}{\phi_0}$$

Therefore,

$$a = \eta_A - \eta_S = \frac{\left(r + \frac{t^2}{12}\right) \sin \phi_0}{\phi_0} - \frac{r \sin \phi_0}{\phi_0} = \frac{t^2\,l}{12\,L}$$

and,

$$f = \frac{12\,I_x\,L}{t^2\,l \int_0^{\phi_0} d\,g} = \frac{I_x\,E\,t^3}{t^2\,l} = \frac{6\,r^2\,K_0}{t^2}$$

Introducing these values,

$$\Delta H = \frac{p\,t^2}{6\,r\,K_0} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (94)$$

which is identical with Equation (24).

The real value of the foregoing theorem lies in its application to arches under non-uniform, non-symmetric loads. ΔH, therefore, no longer acts along the x-axis, but lies obliquely to it, producing a component in both the x and the y-axes. A graphical analysis will then give quickest results. Such an analysis is shown in Fig. 20. The arch is divided into elements of equal and

finite length and their centers of gravity, elastic weights, Δg, ρ, and ρ', are determined. The center of gravity of the elastic weights of the whole arch is then determined and through it, the x and y-axes as principal axes of inertia of the arch. Major and minor axes of the ellipse of elasticity of the whole arch are then computed and plotted, and the ellipse is drawn, thereby establishing a scale of the elastic properties of the arch which are constant for one and the same arch and, therefore, are not affected by any load on the arch.

The elemental division lines are now extended into the load area, and the elemental loads are computed and combined in a convenient scale with an arbitrary pole to a force polygon. The corresponding funicular polygon is then drawn in any position, observing, however, for the sake of simplicity, that it does not intersect the arch axis. After determining the antipoles of the forces (line of pressure), with respect to the elemental ellipses of elasticity, and letting the elemental rotations, $\Delta\delta = N s \Delta g$, act at the antipoles as weights parallel to both the x and the y-axes, their center of gravity, A, may be found. As A, also, is the antipole of ΔH with respect to the ellipse of elasticity of the whole arch, the position of ΔH is at once determined and its magnitude is:

$$\Delta H_N = \frac{\Sigma \Delta \delta}{f \Sigma \Delta g} \qquad (95)$$

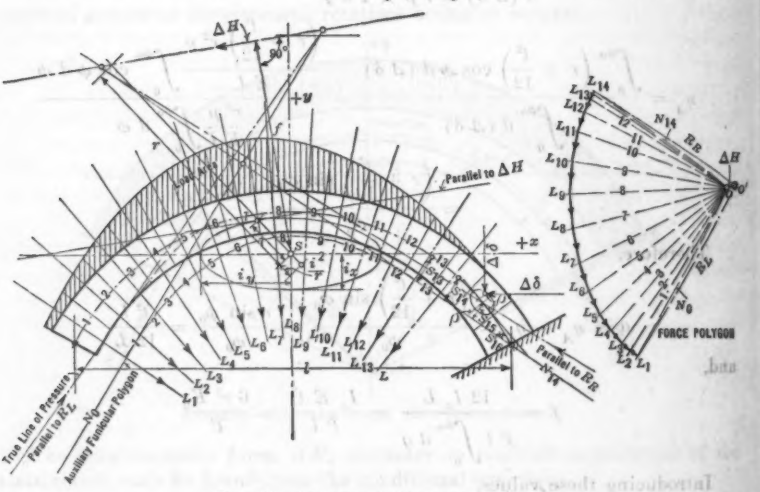

FIG. 20.—GRAPHICAL ANALYSIS OF ARCH.

By plotting ΔH from Pole O negative, that is, contrary to its direction of action, the true pole, O', is established. The two force polygons, the temporary and the true one, are affined with ΔH as their axis of affinity. The true line of pressure, therefore, is found by remembering that corresponding rays (forces) intersect on the axis of affinity. For a change of temperature of $\pm \Delta T°$ Fahr.:

$$\Delta H_T = \frac{\pm \Delta T \alpha E l}{\Sigma \Delta g i_x^2} \qquad (96)$$

For relatively thin arches it is permissible to use $I_x = \Sigma y^2 \Delta g$ and $I_y = \Sigma x^2 \Delta g$, whereby a material reduction in computation is effected. This approximation gives somewhat smaller values for I_x and I_y and, therefore, is on the side of safety.

A. Floris,* Esq. (by letter).—The method of determining stresses in a dam by using the conception of combined cantilever action and arch action is comparatively well known. One investigation, dealing with the arch crown alone, dates back to 1889,† and several attempts have been made to determine the stresses in other points besides the crown. The author's method of analysis was applied to the design of the La Jogne Dam‡ in Switzerland, and to several other dams§ of the U. S. Bureau of Reclamation.

Professor C. Guidi also gives several sets of equations‖ for the determination of stresses at various points of the arch, using the cantilever and arch action theory. He states, however, that although the solution of these equations presents no special difficulty, they are of little practical value because the number of the unknowns to be determined is too great. Consequently, in order to simplify the analysis, Professor Guidi has assumed the distribution of the water pressure to vary according to a parabolic law with a full water pressure at the abutments, diminishing to three-quarters of this pressure at the crown. The cantilever action is entirely neglected in this case and the elementary arch is analyzed as an independent unit. This assumption is based on the results of his theory¶ and tests‖ in the case of a dam site with a rectangular shape.

The author's trial-load method seems quite laborious as indicated by the incomplete analysis presented in the paper. The lack of detail is a great disadvantage to those who desire to follow the method closely. On the other hand, complete derivations of some of the formulas presented have been published by Professor Guidi.

The practical importance of these formulas, based on the theory of the ellipse of elasticity, may be questioned when compared with other well known analytical methods.

The author also submits Professor Guidi's diagrams** (modified and improved), for the determination of the stresses in an arch of constant thickness due to uniform water pressure and to a change in temperature. As a means of determining the effect of non-uniform water pressure on the stresses, the author has extended Professor Guidi's influence lines†† to cover some special

* Los Angeles, Calif.

† "On the Strains in Curved Masonry Dams," H. Vischer and L. Wagoner, *Transactions*, Technical Soc. of the Pacific Coast, December, 1889, p. 75.

‡ "Etude sur les barrages arqués," A. Stucky, *Bulletin technique* de la Suisse romande, 1922.

§ "Analysis of Arch Dams by the Trial Load Method," C. H. Howell, M. Am. Soc. C. E., and the late A. C. Jaquith, Esq., see p. 1191.

‖ "Studi sperimentali su costruzioni in cemento armato," C. Guidi, Estratto dal rendiconto dell' XI Riunione dell' Associazione Italiana per gli studi sui materiali da costruzione, Turin, 1926, p. 18; or its French translation, "Etudes expérimentales sur les constructions en béton armé," A. Paris, *Bulletin technique* de la Suisse romande, 1927, p. 19; "Esperienze termiche su di una diga a volta," C. Guidi, Estratto dagli Atti della Reale Accademia delle Scienze di Torino, Vol. LXII, 1927.

¶ "Statica delle dighe per laghi artificiali," C. Guidi, Turin, 1921, p. 71; and "Sulle dighe ad arco," C. Guidi, *Estratto da L'Ingegnere*, Vol. I, August, 1927.

** "Statica delle dighe per laghi artificiali," C. Guidi, Turin, 1922, Appendix, pp. 6 and 7.

†† *Loc. cit.*, p. 70.

cases. This is perhaps the most valuable part of the paper, which will serve, somewhat, to familiarize American engineers with Professor Guidi's scientific investigations on the design of arch dams.

FREDRIK VOGT,* ASSOC. M. AM. SOC. C. E. (by letter).—Under "Proposals for Improvements", the author has emphasized varying the radius of curvature as the best way to make the arch better able to sustain the non-uniform load to which it is subjected. He refers to the Italian Government regulations which state that it is desirable to vary the curvature as may be required in order to maintain the thrust axial; but these regulations give no indication as to how this should be done.

In fact, it is quite impossible to solve the problem of maintaining the thrust axial and thereby to equalize the arch stresses by varying the curvature. The stress distribution may be improved in that way, but the stresses cannot be equalized as assumed in the Italian rules. Imagine that, for a certain shape, the thrust due to water pressure is axial at all points of the arch, and that no stresses existed before applying the load. With this axial thrust the arch would be compressed but not bent; the total length of the chord would thus be reduced. To make the arch fit the abutments a rib-shortening pull must be applied. This force is eccentric at all except two points of the arch and, therefore, it produces bending moments and bending stresses. It seems as if this well-known fact is overlooked sometimes.

In order to obtain axial thrust and uniform stress distribution under load, it is necessary to apply initial stresses in the arch. Heating the arch would do that in an ideal manner, but that is, of course, out of the question. Swelling of the concrete also would do it. However, the fact that the contraction joints in dams can be grouted after some time shows that the shrinkage due to chemical action in the concrete and to cooling from the high setting temperature more than equalizes the swelling due to water-soaking; the final result is shrinkage even after the dam is entirely water-soaked. The cracks in many dams without joints show that the resulting shrinkage is usually greater than the extensibility of the concrete in tension. This condition means that the non-uniformity of the arch stresses is increased.

The question of how to equalize these arch stresses is of great importance since a thin arch with uniform stress distribution can carry the same load as a thicker arch with non-uniform distribution. The efficiency of arches will be highly increased if the stresses can be equalized and thereby a saving in material can be attained.

The writer can see no other way to obtain initial stresses for equalization of the final stresses than by use of a multiple-pressure grouting.† A division both vertically and horizontally of some of the contraction joints into compartments and the grouting of the different compartments under different pressures make it possible to obtain initial stresses distributed in about the same way as the stresses due to a heating of the dam. By means of such initial stresses in the arch not only can the arch stresses under load be approx-

* Cons. Engr., U. S. Bureau of Reclamation, Denver, Colo.

† As outlined in his discussion of the paper entitled "Analysis of Arch Dams by the Trial Load Method," by C. H. Howell, M. Am. Soc. C. E., and the late A. C. Jaquith, Esq., see p. 1272.

imately equalized, but the division of load between arches and cantilevers can also be checked in order to reduce the cantilever load as much as may be found convenient.

F. H. CONSTANT,* M. AM. SOC. C. E. (by letter).—This paper is a continuation of the subject of the solution of arch dams by the trial load method, based on equal deflections of the horizontal arch and the vertical beam elements at every point of the dam. The method described was first suggested by William Cain, M. Am. Soc. C. E.† It was later developed by F. A. Noetzli, M. Am. Soc. C. E.,‡ for the crown section only, under the assumption that the water load is uniformly distributed on each horizontal arch, and that the vertical beam is a cantilever fixed at the base. While the Noetzli method is also applicable to the case in which the vertical beam is hinged at the base, the additional condition introduced—that of zero moment at the base—greatly complicates the solution by trial. It may almost be said at the outset that the use of the trial method pre-supposes a dam with a fixed base. It can be expected, when the necessity for so doing is recognized, that efficient means for anchoring the dam to the bed-rock will be devised.

It was apparent immediately after Mr. Noetzli's paper was published, that there is no basis for the assumption of a uniform water load, and that the actual distribution cannot be determined by a study based merely on a coincidence of deflections at one section only. The paper§ by C. H. Howell, M. Am. Soc. C. E., and the late A. C. Jaquith, Esq., which met these defects squarely, and which demonstrated the practicability of the trial method in actual design, was "a long step forward". Mr. Sutherland's paper is a continuation of the same theme, and as, in the writer's opinion, the trial method, based on coincident deflections at every point of the dam, is the only practical solution of the problem, it is very welcome. Each contribution along this line helps to make clear some of the uncertainties and offers valuable suggestions to the designer. What is needed now is a definite outline of procedure, absolutely reliable formulas, reduced to their simplest terms and free from integral and other operating signs, and some preliminary idea of the water load distribution on the horizontal arches.

Each of the last two papers mentioned, gives valuable information in regard to water load distribution for actual dams; that by Mr. Sutherland, in particular, for the Salmon Creek Dam. It is interesting to note that he finds it is nearly uniform near the top (at Elevation 1 151), but that it becomes parabolic about one-third the way down (Elevation 1 103). Messrs. Howell and Jaquith also obtained a nearly uniform distribution on the top arch. The author states: "It will be found that, for the lower arch sections, the load falls away rather rapidly from the center, whereas for the higher arch sections the loading varies less from a uniform distribution". This is valuable information for preliminary trials of future arch dams. More data of this kind are needed.

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* Prof. of Civ. Eng., Princeton Univ., Princeton, N. J.

† *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 2084.

‡ "Gravity and Arch Action in Curved Dams," *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 1.

§ "Analysis of Arch Dams by the Trial Load Method," see p. 1191.

On account of the variation of the water load between the top and the lower arch sections, only formulas that will give influence deflection curves are of value. The author develops such formulas in Appendix II and has translated them into curves (Fig. 15), which (for the data on which the latter are based), are very valuable. Ultimately, it may be desirable to prepare a comprehensive set of such curves covering a wide variation of data, in order that the designer may be able to select the necessary influence lines from the curve without the labor of computing them from the formulas. The explanation of the use of curves is one of the valuable contributions of this paper.

The writer has previously given formulas for the horizontal thrust and bending moment at the crown and radial deflection at any point of an arch that is fixed at the ends and subjected to a pair of symmetrically placed radial unit loads.* They were applied to the Stevenson Creek Dam to obtain influence values for deflections at each 10-ft. point for each 10-ft. horizontal section between the crest and the lowest point of the dam. A few experienced computers, using a calculating machine, and systematically tabulating results, can get all these values in two or three days.

The influence deflections thus obtained† were found to satisfy Maxwell's principle of reciprocal deflections fairly closely; that is, having two deflection influence curves—for example, one for the 10-ft. point and the other for the 30-ft. point of an arch—the y-ordinate at the 30-ft. point of the first curve was equal to the y-ordinate at the 10-ft. point of the second curve.

The problem of the arch dam is still further complicated by the effects of temperature variation, shrinkage, etc., and much more study must be given to it before a final method of design is reached.

R. A. SUTHERLAND,‡ ASSOC. M. AM. SOC. C. E. (by letter).—Speaking of plastic flow, Professor Cain has stated that "this is a consoling theory, but its realization was not much in evidence at the Stevenson Creek Experimental Dam * * *". The rate of loading in those experiments was, relatively speaking, instantaneous, and cracking was to be expected. In the "high" arch dams (more than, say, 150 ft.) with which this paper dealt, the rate of loading is generally slow, and plastic flow is a factor which must be considered. The writer's method of computing stresses under two extreme conditions corresponds with assuming: First, that the beams or cantilevers are cracked; and, second, that they are not cracked. By taking mean stresses a reasonable compromise is effected, which is the most that can be done at present. No theory can fit all the facts when dealing with such an imperfectly elastic material as concrete.

Professor Cain has shown previously§ that at one point on any assumed beam "all the water pressure is carried by the arch and none by the cantilever." The existence of such a point is, however, not incompatible with Figs. 5 or 18. It should be emphasized that the "crest reaction" load shown in Figs. 4, 5, and 18 is not an additional load on the dam, but simply part of the water load

* See p. 1242; see, also, discussion by William Cain, M. Am. Soc. C. E., p. 1265.
† See p. 1242, Equation (61).
‡ Chf. Engr. and Asst. Mgr., Christmas Island Phosphate Co., Christmas Island.
§ See p. 1265.

on the vertical beam transferred as a reaction to the crest support. To complete the load diagram, as far as the vertical beam is concerned, a horizontal arrow should be inserted at the base, directed up stream, to represent the remaining beam reaction and equalize the forces acting up stream and down stream. The effect of the crest reaction shown in Figs. 4, 5, and 18, is that, near the crest, certain loads due to part of the water load are forcing the beam down stream, while other loads due to the interaction of beam and arch are forcing the beam up stream. At one point, these forces will be equal, and the net force on the beam will be nil. This point will correspond with Professor Cain's point, *K*.*

Mr. Jakobsen points out that the formulas given by the writer are correct only for relatively thin arches, such as $\frac{t}{r}$ less than 0.3. This does not prevent their use for the majority of arch dams with relatively small error. It can easily be shown that the condition, $\frac{t}{r} < 0.3$, is very nearly equivalent to the condition that the depth is less than $\frac{3}{8} f$, in which, f is the cylinder stress, in pounds per square inch. Taking a usual value of 300 lb. per sq. in. (about 20 long tons per sq. ft.), this condition holds for plain arch dams up to 180 ft. high, and for higher constant-angle arch dams. For dams of greater height arch action is small in the lower part and errors made will be relatively unimportant even if $\frac{t}{r}$ is excessive.

Mr. Jakobsen has shown that the writer's assumption of zero arch load at the abutments is incorrect. This is an important point in the theory of arch dams in so far as the resultant arch stresses at the abutments are concerned. It will not, however, have an appreciable effect on the deflections, on which the writer's method of load distribution depends, because the arch deflection ordinates fall away to zero so rapidly at the abutments. This will be clear from Fig. 5 (*d*), (*e*), (*j*), and (*k*), in which the arch deflection curves would be scarcely altered even if the arch load were given an appreciable value instead of zero at the abutment. The result on the arch stresses will be more important and will tend to increase the intradosal stresses at the abutment to more than those found by the writer.

The writer does not agree with Mr. Jakobsen that shrinkage has been neglected in the paper, since the manner of allowing for this was fully dealt with in the section entitled "Temperature and Shrinkage". Since nearly all large arch dams are now built with vertical joints, shrinkage stresses are less important than in the case of a small dam like that of Stevenson Creek.

Mr. Bauman's interesting and constructive discussion emphasizes the utility of the ellipse of elasticity in the analysis of thrusts and deflections due to non-uniform and non-symmetrical loads. He also shows that the cost of reinforcing steel required to take such tensile stresses as do occur in an arch dam is a very small percentage of the total cost. In spite of this the practice of reinforcing large arch dams has seldom been followed, probably on account of

* *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 74, Fig. 15.

the difficulty of anchoring the bars adequately into the foundation, and of avoiding interference with the pouring of the concrete in bays. Most investigations on arch dams have aimed at devising methods of avoiding or reducing tension rather than accepting and providing for it, but the writer believes that the possibility of reinforcing arch dams might be profitably discussed.

Dr. Vogt states that the system of multiple-pressure grouting appears to be the only method of equalizing final arch stresses. This method would give ideal results for only one given set of conditions of water load and temperature, but it appears to hold out considerable promise in the direction of increased economy and security of arch dams.

The writer agrees with Professor Constant that the trial-load method should be founded on the assumption of fixed base. While the elastic yielding of the foundation rock is equivalent in some measure to pivoting at the base, the effect of such action is to reduce beam loads, and hence is on the side of safety.

Professor Constant puts forward a plea for "some preliminary idea of the water load distribution on the horizontal arches". The writer believes that this will be done in time, by determining the distribution for various standard profiles of site, such as were referred to in the writer's discussion of the Stevenson Dam tests.*

In fact, once a method of procedure is adopted as satisfactory, there is no reason why the design of arch dams should not be standardized for various site profiles, heights, and crest spans, the designs, of course, being based on a certain theoretical stress. This would entail an enormous amount of work, but would be of great benefit in the preliminary design of dams, and would make immediately available to all engineers the results of the research work which has been done by various organizations with extensive resources.

* *Proceedings*, Am. Soc. C. E., November, 1928, Papers and Discussions, p. 2610.

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Paper No. 1720

SILTING OF THE LAKE AT AUSTIN, TEXAS*

By T. U. Taylor,† M. Am. Soc. C. E.

With Discussion by Messrs. L. M. Lawson, C. H. Eiffert, Oren Reed, Charles Schultz, John B. Hawley, H. F. Robinson, Julian Montgomery, Kirk Bryan, J. C. Stevens, P. A. Welty, Banks McLaurin, R. G. Tyler, E. C. H. Bantel, R. F. Walter, William Jenner Powell, R. I. Meeker, and T. U. Taylor.

In 1890, cross-sections of the old lake formed by the Austin Dam were taken at different points, as shown in Fig. 1 and in Table 1. The crest of the old Austin Dam was 60 ft. above the toe, or apron of the down-stream face, whereas the overflow crest of the new dam is 9 ft. lower, or 51 ft. above the toe.

The water first flowed over the crest of the old dam in May, 1893, and there were at this time 79 537 303 cu. yd., or 49 300 acre-ft. of water in the lake, rising to the level of the crest of the old dam. In February, 1900, there were 41 528 810 cu. yd., or 25 741 acre-ft. Thus, the water capacity of the lake in February, 1900, was only 52% of the capacity of that in May, 1893. If these volumes are estimated in depths on a square-mile base, the result is, in 1893, a volume of water equal to a depth of 78.6 ft. and, in 1900, an equivalent depth of 38.4 ft. of this was silt. This was the result from May, 1893, to January, 1900, a period of 6.75 years.

Table 1 shows the maximum and mean depths of water for 1893 and 1900; the maximum and mean depths of silt for 1900; and the percentage of silting at the respective stations.

The maximum depth of silt is not always equal to the differences of the maximum depths of water for 1893 and 1900 because the silt has shifted at several points. This was very noticeable at Station 12.525, known as Santa Monica, or Sulphur Springs. The column of percentages is the ratio that the cross-section of 1900 (of silt) bore to the original cross-section of water.

* Published in February, 1928, *Proceedings*.

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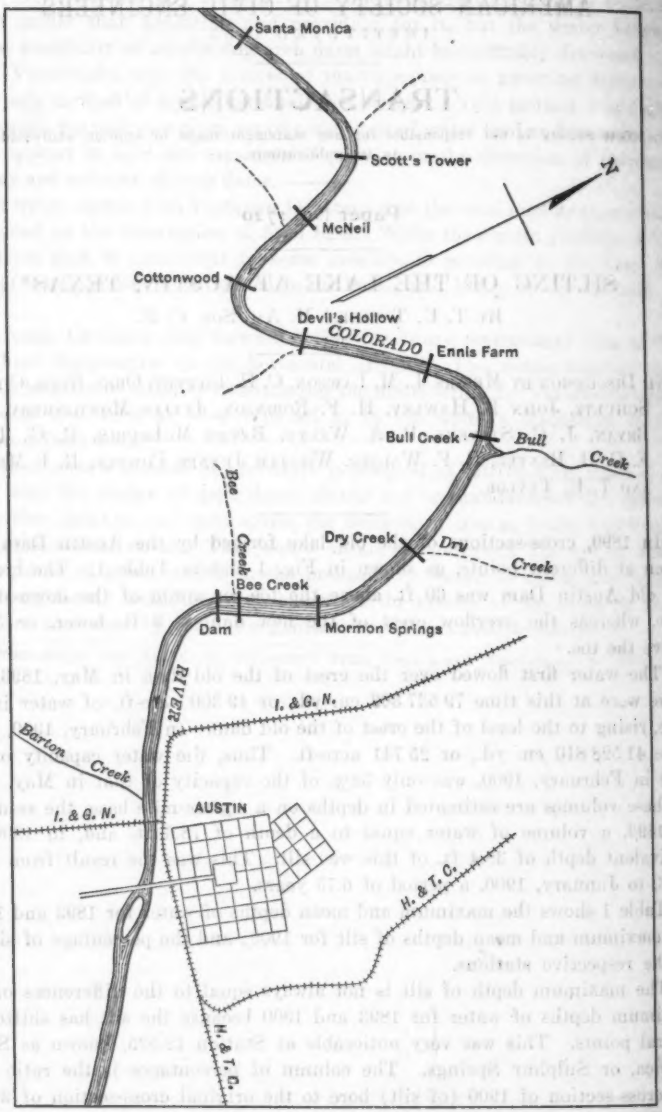


FIG. 1.—MAP SHOWING LOCATION OF AUSTIN DAM.

Thus, at Santa Monica Springs (Station 12.525), 78.9% of the original cross-section has been filled in 6.75 years.

Between Stations 0 and 17.75, the silt in 1900 was a fine, impalpable, absolutely gritless deposit. From Station 17.75 to the head of the lake, there was a distance of nearly 3 miles where the silt consisted largely of sand.

TABLE 1.—Depths of Water and Silt in the Reservoir Formed by the Old Dam at Austin, Texas.

Maximum Depth.

Distance of station, in miles, from dam.	Depth of Water, in Feet.		Depth of Silt, in Feet.
	1893.	1900.	1900.
0.0	66.0	39.0	27.0
0.221	65.7	38.0	27.7
1.173	63.8	35.5	28.3
2.698	56.0	31.5	24.5
4.012	47.8	31.5	19.0
5.337	47.5	27.5	23.0
6.799	47.0	22.0	26.0
7.810	44.8	20.5	25.0
9.189	40.4	13.4	29.0
10.221	40.9	13.5	27.4
12.525	29.4	9.8	26.0
13.425	24.0	12.5	16.0
14.725	16.6	12.0	15.0
16.225	13.2	9.5	7.0
17.725	3.7	2.2	1.5

Mean Depth.

Distance of station, in miles, from dam.	Mean Depth of Water, in Feet.		Mean Depth of Silt, in Feet.	Amount of fill, percentage of total.
	1893.	1900.	1900.	
0.0	40.1	27.5	12.6	31.2
0.221	37.6	26.5	11.1	32.2
1.173	30.6	21.7	8.9	20.2
2.698	33.4	21.5	11.9	38.8
4.102	38.0	23.5	14.5	37.9
5.337	36.7	23.0	13.7	42.0
6.799	36.3	17.4	18.9	56.3
7.810	30.7	15.4	15.3	56.3
9.189	30.8	10.2	20.6	66.9
10.221	27.2	7.9	19.3	71.8
12.525	20.3	5.5	14.8	78.9
13.425	17.2	9.0	8.2	50.4
14.725	13 2	10.0	3.2	60.0
16.225	11.2	8.8	2.4	30.0
17.725	5.6	3.9	1.7	33.0

The Pedernales River traverses sandy valleys and enters the Colorado River several miles above the head of the old lake, and at flood stages it is heavily charged with sand. When these flood waters strike the relatively still waters of the lake and the velocity is checked, this heavy sand is deposited. At occasional points below the head of the lake, small bars of sand were found near the mouths of small canyons or creeks. The cross-sections of the old lake are shown in Fig. 2. The vertical scale is ten times the horizontal. (See, also, Table 2).

From March 15 to April 17, 1899, the water level of the lake was a little more than 10 ft. below the crest of the dam. The water again commenced to overflow on April 21, 1899, when the river rose to a height of 9.8 ft. above the old crest. This flow continued until June 2, 1899, and its result on the cross-sections near the head of the lake was marked. The sections at Stations 13.425, 14.725, 16.225, and 17.725 were scoured out from 2 to 3 ft. deeper than the sections of 1897, and, at Station 14.725, a sand-bar was deposited on the inside (left) of the curve of the river, which contracted the channel to less than one-half its former width.

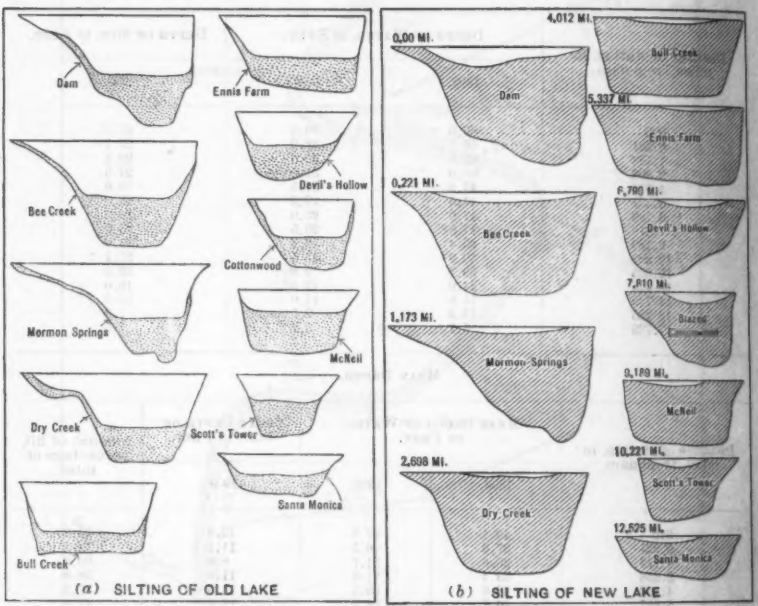


FIG. 2.

From above Colorado City, Colo., on the Texas and Pacific Railroad, the river flows through a hilly country for 300 miles, and absorbs in its course the waters of the Conchos, the San Saba, the Llano, and Pecan Bayou, as well as those of the Pedernales. All the country drained by these tributaries is hilly, with the exception of a few miles along the head of the Colorado and the Conchos Rivers.

SILTING OF NEW LAKE AUSTIN

The contract for the present Austin Dam was let in 1911, and the dam was partly completed by the summer of 1913. In September of the same year, the lake was filled with water up to the present spillway, which is 51 ft. above the toe of the old dam. This datum was the one usually adopted. The

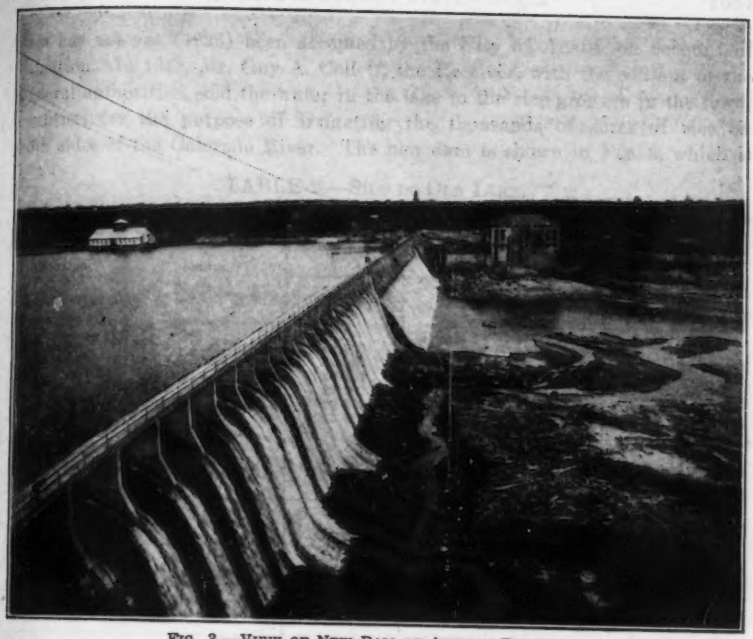


FIG. 3.—VIEW OF NEW DAM AT AUSTIN, TEXAS.

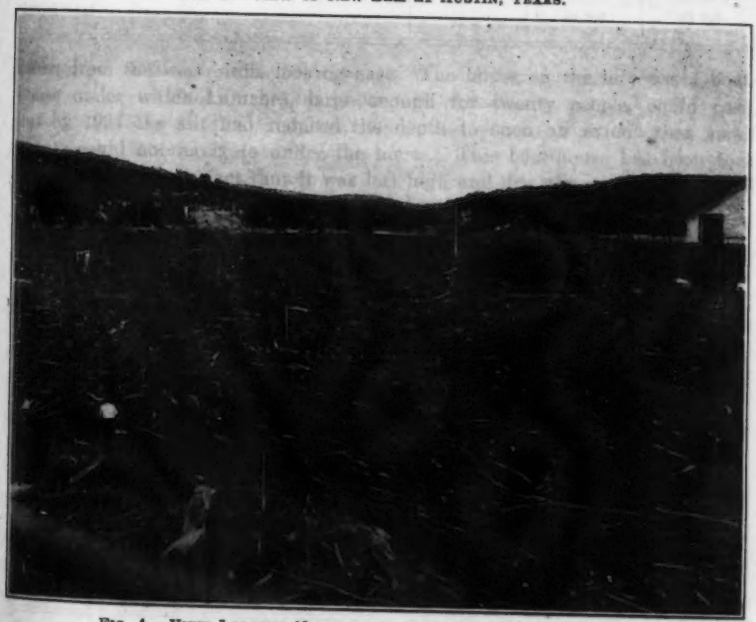
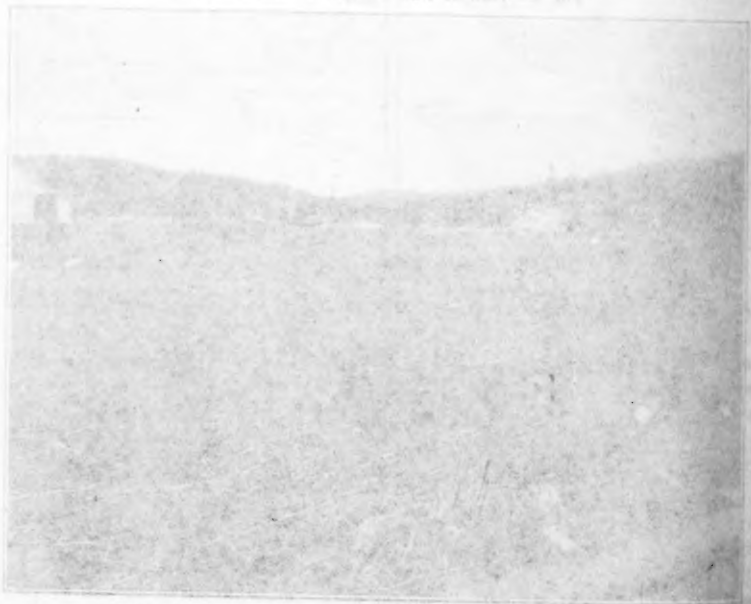


FIG. 4.—VIEW LOOKING NORTHWEST FROM THE NEW AUSTIN DAM.



dam has not yet (1926) been accepted by the City of Austin on account of litigation. In 1918, Mr. Guy A. Collett, the Receiver, with the consent of the Federal authorities, sold the water in the lake to the rice growers in the lower counties, for the purpose of irrigating the thousands of acres of rice on both sides of the Colorado River. The new dam is shown in Fig. 3, which is

TABLE 2.—SILT IN OLD LAKE.

Station.	Name.	Area, in Square Feet.		Volume of Water, in Acre-Feet, Below Crest of Dam.	
		1893.	1900.	1893.	1900.
0.0	Dam	48 000	33 000	1 286	878
0.221	Bee	48 000	32 500	5 100	3 504
1.173	Mormon	40 400	28 200	7 316	4 796
4.012	Bull	32 980	20 480	5 419	3 250
5.337	Ennis	34 500	20 000	5 531	2 853
6.799	Devils	27 925	12 200	3 010	1 315
7.810	Cottonwood	21 200	9 265	3 727	1 422
9.189	McNeill	23 400	7 750	2 519	782
10.221	Scotts	16 870	4 750	4 757	1 170
12.525	Santa Monica	17 200	3 630	.....	.....
Sub-total		.....	.....	44 377	23 488
13.425		11 180	5 500	1 548	498
14.725		7 700	3 300	1 487	670
16.225		5 100	3 500	1 163	591
17.725		2 880	1 940	725	494
Grand total		.....	.....	49 300	25 741

taken from the west bluffs looking east. The house on the left was a boat-house under which launches, large enough for twenty people, could pass, but by 1924 the silt had reduced the depth to such an extent that small canoes could not navigate under the house. This boat-house has been torn down because of the fact that it was left high and dry, the silt having forced

TABLE 3.—SURVEY OF LAKE AT AUSTIN, TEXAS, AUGUST, 1926.

Distance of station from dam, in miles.	Water Areas, in Square Feet, Below Crest of Dam.			Water Volume, in Acre-Feet, Below Crest of Dam.			
	1913.	1922.	1924.	1913.	1922.	1924.	1926.
0.0	36 000	13 000	11 600	981	322	249	97
0.221	37 300	11 100	6 980	3 790	982	520	224
1.173	28 400	5 980	2 040	5 037	920	316	143
2.698	26 100	3 080	1 380	4 000	470	224	86
4.012	24 130	2 870	1 430	4 009	469	227	108
5.337	25 800	2 970	1 390	4 075	582	292	142
6.799	20 200	3 600	1 900	2 450	357	250	161
7.810	19 800	2 230	2 186	3 000	382	308	189
9.189	16 100	2 350	1 460	1 695	310	187	88
10.231	11 000	2 610	1 530	2 988	568	328	239
12.525	10 400	1 460	820	.....	....	....	....
Total volume, in acre-feet	.....	.....	.....	32 025	5 362	2 901	1 477

the water farther toward the main channel to the west. Fig. 4 is a view looking west from the top of the new dam.

During the latter part of the summers of 1922, 1924, and 1926, the writer made measurements of the cross-sections of the present lake at the old stations, and found the results to be as given in Table 3, and as illustrated by the cross-sections in Fig. 2(*b*).

Theory of Silting of Lake Austin

The silting of reservoirs for purposes of comparison is best reduced to heights or depths on a square-mile base. To derive an expression for the quantity of silt deposited in a given time, let x = the depth, in feet, of silt deposited by each foot in height per year; and h = the original depth of water in the reservoir. The depths of water at the end of 1, 2, 3, and n years are:

At the end of 1 year, depth of water = $h\ (1-x)$
At the end of 2 years, depth of water = $h\ (1-x)^2$
At the end of 3 years, depth of water = $h\ (1-x)^3$
At the end of n years, depth of water = $h\ (1-x)^n$

Hence, if d = the depth of water at the end of n years,

$$d = h\ (1-x)^n \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

This will not be true for any reservoir in which there is an appreciable current acting on the bottom of the basin, that is, on the upper surface of the silt. In the old lake at Austin, the lake level could sink to 10 ft. below the crest of the dam, and still have a fair current in the penstocks, as the bottom of the forebay was 12 ft. below the crest of the old dam.

The measurements on the new lake were carried no farther up the lake, for the reason that the next station above Santa Monica was filled entirely with sand. In fact, sand was encountered about ½ mile below Santa Monica, or at a point 12 miles from the dam. Table 3 includes only the fine gritless mud. The volume of water in:

1913 = 32 029 acre-ft.
1922 = 5 362 acre-ft.
1924 = 2 901 acre-ft.
1926 = 1 477 acre-ft.

If these are reduced to the depths, in feet, on a square-mile base, the volume of water in:

1913 = 50.05 ft.
1922 = 8.38 ft.
1924 = 4.53 ft.
1926 = 2.31 ft.

$$\text{Ratio of water volumes} = \frac{\text{Volume of 1926}}{\text{Volume of 1913}} = 0.0461 = 4.61\%$$

Thus, in the thirteen years the Austin Lake has filled 95.39% of its volume as measured in 1913. The water has now (1926) 4.61% of the volume it had in 1913.

In Equation (1), $h = 50.05$, and the depth of water = 2.31. Hence,

$$50.05\ (1 - x)^{11} = 2.31$$

and,

$$x = 0.20$$

That is, on an equal base, an average of 20% of silt was deposited each year. Fig. 5 illustrates the relative quantity of silt and water in September, 1926.

Drift

The Colorado River of Texas, with its tributaries, traverses nearly 2 000 miles of wooded territory, and every rise brings its sinuous ribbon of drift, which ranges in width from 40 to 200 ft. When the first drift strikes the gates or buttress walls of the Austin Dam, it spreads out over the entire width of the lake, and banks up behind, until the complete surface of that part near the dam is covered with drift from shore to shore. At times, this covers an area more than ¼ mile wide and ½ mile long.

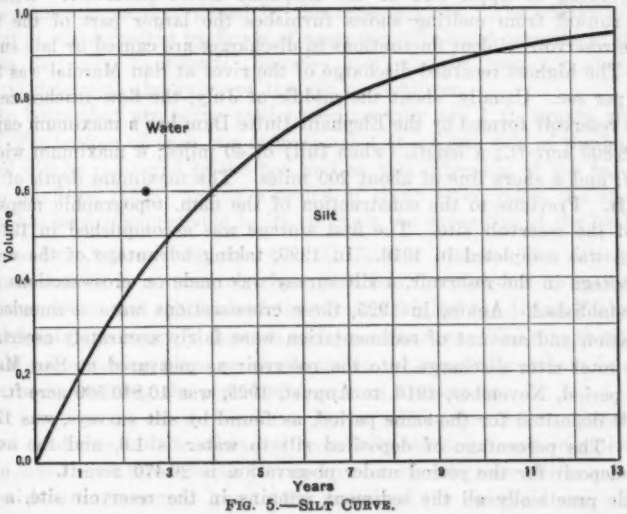


Fig. 5.—Silt Curve.

Fig. 4 was taken from the top of the dam, looking northwest. A line, joining the man in the foreground with the building on the right, points straight up the river. This drift sometimes covers an area of 80 acres, and the velocity of the water in the lake gradually compacts the drift toward the dam, making it more and more dense.

As the water recedes and the lake level sinks, the part of the drift between the man and the power-house rests on the alluvial soil beneath—the result being that the silt near the dam is in reality mud reinforced with sticks, débris, and all manner of floating logs.

In the background is noticed a strip of driftless water. This water is in Bee Creek, above which the silt is almost free from drift, because the velocity of the water in the lake compresses the drift area into that section between the creek and the dam.

DISCUSSION

L. M. LAWSON,* M. AM. SOC. C. E. (by letter).—The author has described one of the important features of the storage of water in the Southwest where inflows carry a large percentage of sediment. An opportunity for similar study presented itself in the creation of the Elephant Butte Reservoir on the Rio Grande. The Elephant Butte Dam forms one of the largest reservoirs in the United States for irrigation, and during the writer's superintendency of the Rio Grande Project (of which this dam is the main storage feature), two silt surveys were made and interesting data collected on the amount of sedimentation.

The Rio Grande, which is the source of the water supply for the Rio Grande Federal Irrigation Project, rises in Southern Colorado and has a drainage area of 30 000 sq. miles above San Marcial, N. Mex., which point is the beginning or upper end of the Elephant Butte Reservoir. While the spring run-off from melting snows furnishes the larger part of the inflow into the reservoir, violent fluctuations in discharges are caused by late summer rains. The highest recorded discharge of the river at San Marcial was 33 000 cu. ft. per sec. Usually, about the middle of July, the flow reaches zero.

The reservoir formed by the Elephant Butte Dam has a maximum capacity of 2 638 800 acre-ft.; a length (when full) of 40 miles; a maximum width of 6 miles; and a shore line of about 200 miles. The maximum depth of water is 200 ft. Previous to the construction of the dam, topographic maps were made of the reservoir site. The first storage was accomplished in 1915 and the dam was completed in 1916. In 1920, taking advantage of the unusual large storage in the reservoir, a silt survey was made on cross-sections previously established. Again, in 1925, these cross-sections were re-sounded and the location and amount of sedimentation were fairly accurately ascertained.

The total river discharge into the reservoir as measured at San Marcial, for the period, November, 1916, to August, 1925, was 10 840 500 acre-ft. The total silt deposited for the same period, as found by silt surveys, was 177 740 acre-ft. The percentage of deposited silt to water is 1.6, and the average annual deposit for the period under observation is 20 470 acre-ft.

While practically all the sediment remains in the reservoir site, a large percentage is deposited above the ordinary usable storage elevation.

C. H. EIFFERT,† M. AM. SOC. C. E. (by letter).—The rate at which the capacity of the Austin Reservoir has been reduced is truly remarkable. In the retarding basins of the Miami Conservancy District the rate of silting is comparatively very small.

The manner of silting in these basins will differ greatly from that described by Dean Taylor. Water is stored only temporarily during floods. Most of the time they are entirely empty, and practically all the included area is good agricultural land and is farmed every year. To this farm land the deposit of silt is of great value as fertilizer.

* Commr., U. S., International Boundary Comm., El Paso, Tex.
† Chf. Engr., The Miami Conservancy Dist., Dayton, Ohio.

The annual rate of deposit will not be uniform because the number of floods and their intensity differ considerably from year to year. The amount of silt carried will undoubtedly vary quite a little according to the season at which a flood occurs and also according to the intensity of the rainfall and the condition of the soil and crop growth at the time of flood. During the intervals between floods there will be a great many local rains, which, while they will not cause back-water in the basin, will result in considerable run-off and local erosion, thus tending to remove much of the silt deposited by back-water.

In two of the basins where the rate of silting is likely to be greater than in the others, a number of cross-sections have been taken. This work will be repeated from time to time and the amount of silt remaining as a permanent deposit will be determined. In addition, a number of silt-catching boxes have been installed. By means of these it is intended to obtain the actual amount of silt deposited by individual floods for comparison with the amount remaining permanently as determined by the cross-sections. The variation in the amount of silt deposited by floods of approximately equal severity at different seasons may also be found. Samples may also be used for analyses in order to determine the value of the silt as fertilizer.

The boxes that are being used experimentally are 1 ft. square and 6 in. deep. The bottom of each box extends out 1 ft. on one side of the box. The deposit will be measured in the box and on this extension of the base.

To this time (1928) during a period of about seven years, the amount of silting has been inappreciable in its effect on the capacities of the basins.

OREN REED,* ASSOC. M. AM. SOC. C. E. (by letter).—The silting of the Austin Reservoir in a relatively short time is a forceful demonstration of the enormous débris and silt load of the rivers draining the Great Plains. The value of a storage reservoir depends on its permanency. The rate at which erosion of soil is taking place on the basin of the supplying stream is a major factor in determining the period of service of a reservoir.

In many cases, as at Elephant Butte, silting has been accepted as a natural condition and its effect has been fully taken into consideration. The loss in investment through the silting up of a storage reservoir is small when compared with the actual loss to the region of the reservoir site. For this reason, it is of paramount importance that, wherever it is possible, the permanence of a reservoir be assured through adequate protection of the stream basin from erosion.

The Colorado River (of Texas) heads in Eastern New Mexico near the Texas line, at an elevation of less than 4 000 ft. It flows southeastward and crosses the Balcones fault zone a short distance above Austin. The Balcones escarpment is the result of an enormous geologic fault that extends across the entire State from the Red River to the Rio Grande. It marks a sudden change from the gently sloping Rio Grande Plain to the eroded plateaus. The clay soil cover of the water-shed supports a fair growth of scrub timber. The head-waters of the Colorado flow in the deeply fissured limestone of the

* With Swedish Gen. Elec. Co., Vesteras, Sweden.

central plateau and are normally clear. They carry, however, during periods of flood, an enormous quantity of silt eroded from the gorges and channels of the small streams which indent the plateau region. The silt burden has been roughly estimated at 1% of its volume.* Wide fluctuations between minimum and maximum flow are caused by periods of intense rainfall.

It is generally accepted that a forest is an excellent agency to prevent soil erosion. The streams in the Western States, that have their sources in the forested mountains, are largely clear, or of prevailing low turbidity, until they enter the treeless plains. The thin soil cover and low rainfall in the Upper Colorado Basin are not favorable to the growth of a forest cover. In many places it is impossible to maintain a sufficient cover of shrubs and grasses to afford even moderate protection against erosion of the soil.

The use of check dams or settling basins on the head-water streams however, would be of great value. Rough alluvial lands which have been injured by gullying during floods, or upon which sand or gravel bars have been deposited, could be flooded by low and inexpensive dams. Most of the débris and silt carried by the stream would be thus deposited by natural elutriation. For several years check dams have been used to control the flood flow of the upper water-sheds of certain torrential streams in Southern California.† This was an adaptation of the Swiss and Austrian method of torrent control by means of these dams. Channel rectification and levee protection in the Upper Rhine Valley were largely futile until such dams were built to lessen the work of erosion.

All protective cover, whether of grass, brush, or forest, on the head-water streams should be carefully maintained and bank protection should be utilized where economically feasible. For the Great Plains, from Texas northward, about the only practicable means of reducing soil erosion and silting of reservoirs and watercourses is to construct and maintain a comprehensive system of check dams on the head-water streams. The flood flow which at present causes enormous damage to property and life at frequent intervals, would be in part absorbed and retained in ground storage. The increased ground storage would be reflected in a higher minimum flow in the rivers.

CHARLES SCHULTZ,‡ M. AM. SOC. C. E. (by letter).—Because of the nature of civil engineering work, it is not often that engineers are afforded the opportunity to secure records, extending over the long periods of time, as given by the author. Certainly, this paper should make engineers consider this feature of design of reservoirs before proceeding with the expenditure of public moneys. Very little has been written on this subject, and to one who has never had occasion to deal with this problem its possible seriousness seems almost unbelievable.

In 1914, the writer designed a dam in McKinney, Tex., to furnish water for a small country club. The dam (100 ft. wide) was located along a deep ravine, having been constructed to hold back 18 ft. of water and to form a

* *Bulletin No. 1430*, U. S. Dept. of Agriculture.
† *Engineering News-Record*, May 13, 1916.
‡ Tulsa, Okla.

lake ¼ mile long. As this was fed by small springs, no measurement was taken of the drainage area, which was flat, rolling, black land country, such as exists in a belt of land 100 miles wide, extending from the Red River to the Gulf of Mexico. This drainage area was approximately 1 000 acres, and in 10 years the run-off carried enough silt to fill the entire lake, which had to be abandoned. The average rainfall was approximately 40 in., and most of the territory was cultivated farm land.

On another occasion the writer constructed a levee in this same county, along Pilot Grove Creek, a tributary of the Trinity River, which has a width between hills of approximately 1 mile. The creek above this point was approximately 15 miles long, and carried so much silt that along the entire bank where the levee was constructed there was a fill of approximately 4 ft. that had been made since cultivation had begun, and stumps were encountered 4 ft. under the surface.

It is true that most rivers do not carry as much free sediment as some of the rivers in Texas, where the rate of rainfall is severe at times and the soil so light as to be easily washed away. However, this paper will remind engineers that these conditions should be studied and taken into account.

JOHN B. HAWLEY,* M. AM. SOC. C. E. (by letter).—This paper is the result of continuous observation and study by Dean Taylor during more than thirty-five years. He has repeatedly addressed Texas societies and associations, calling attention to the element of siltation and its menace to the success of water-storage projects. Data presented in the paper may be confidently accepted as accurate.

The author has not fully described the character of the drainage area of the Colorado River above Austin. Fig. 6 shows this region, with its areal geology, and the drainage area of the West Fork of the Trinity River above the Fort Worth water supply reservoir, "Lake Worth".

The total drainage area of the Colorado above Lake Austin is approximately 38 000 sq. miles; that of the Trinity above Lake Worth is 1 800 sq. miles. The areal geology, cultivated districts, and axial slopes of the two areas are quite similar; Austin being characterized by Comanchean (Lower Cretaceous), Permian, Pennsylvanian, Later Cenozoic, and the "Central Mineral Region" (Archæan), and Fort Worth by Comanchean, Permian, and Pennsylvanian.

On the Austin drainage area, the part designated "Central Mineral Region" (about 3 000 sq. miles) may be considered, for run-off and siltation purposes, as of the same general character as the Pennsylvanian. The upper 10 000 sq. miles (Later Cenozoic) has a mean annual rainfall of only 15 in.; many of the tributary creeks never reach the main stream at all, but lose themselves in sand flats and "sinks". It would seem improper as a siltation feature, therefore, to include this area. After deducting it, 28 000 sq. miles remain, for purposes of this discussion.

The average mean annual rainfall is approximately 24 in. The mean annual silt (for the two periods reported on by Dean Taylor) is about 2 800 acre-ft., or 0.1 acre-ft. of silt per sq. mile per annum. The Lake Worth drain-

* Cons. Engr., Fort Worth, Tex.

age area is 1 800 sq. miles; the mean annual rainfall is (approximately) 28 in.; and the mean annual siltage since 1916, according to the U. S. Geological Survey and Texas Board of Water Engineers (approximately), 0.5 acre-ft. of silt per sq. mile per annum. This result was determined by "core soundings", because only a few original cross-sections of the lake basin were available.

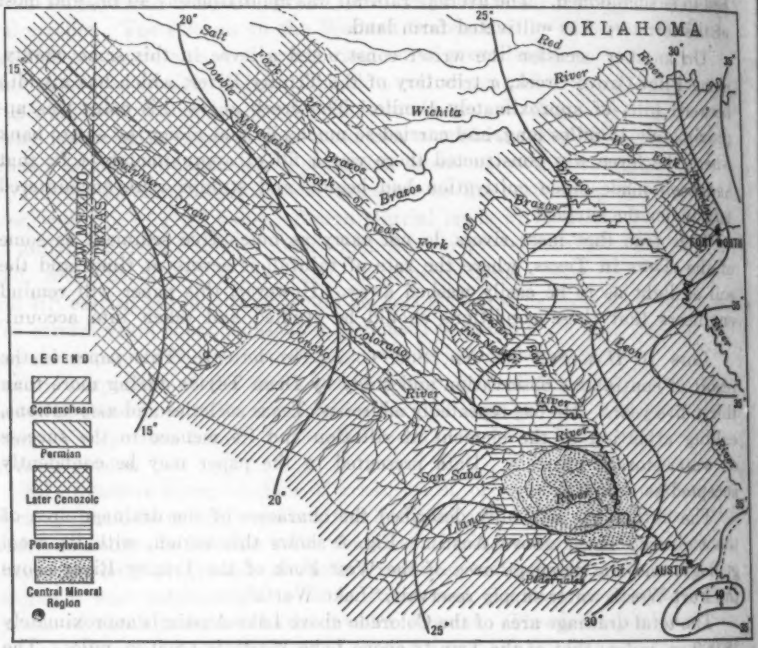


FIG. 6.—DRAINAGE AREA, COLORADO RIVER, ABOVE AUSTIN, TEX., AND OF TRINITY RIVER ABOVE LAKE WORTH.

The difference in character of the areal geology of the two areas is slight; the 4-in. difference in rainfall is scarcely sufficient to account for the vastly larger rate of siltation at Lake Worth. Medina Lake, 40 miles northwesterly from San Antonio, Tex., lies, with its entire drainage area (600 sq. miles), in the Comanchean rocks. Its axial slope is much steeper than that of the Colorado; its mean annual rainfall is about 28 in.; and its rate of siltation, about 0.3 acre-ft. per sq. mile per year. This drainage area is almost contiguous to the southern "divide" of the Colorado.

Personal inspection of substantially the entire Medina water-shed, in 1915, leads the writer to believe that the siltation rate at Medina Lake is naturally much less than at Lake Worth, due to the almost uniformly rocky nature of the Medina area.

Examination of Fig. 2(*a*) indicates that even the old lake merely offered a rather large "channel section" for carrying storm waters, the velocity of which would probably be sufficient to carry a large proportion of fine silt over

the dam. Fig. 2(*b*) clearly shows that current velocities in times of flood were doubtless ample to transport all but the largest and heaviest silt particles over the spillway.

The Austin structure is substantially a "channel dam" and the lake created by it is merely a "channel lake" (Fig. 1). On the Clear Fork of the Trinity, southwesterly from Fort Worth, are three "channel dams", built in 1895. The drainage area above the upper one of these dams is about 500 sq. miles. The geology is Comanchean, with an axial slope of 8 to 10 ft. per mile. The mean annual rainfall is 28 in. The river channel is 26 ft. deep and the dams are 14 ft. high.

The first flood, after completion of these dams, left about 18 in. of silt deposit in each dam. The next flood carried out the silt left by its predecessor, and made a new deposit—and so on to the present time. In 1927—thirty-two years after construction—there is no more silt behind these dams than after the first flood.

In recent years, the writer has observed, at intervals, the discharge of the Colorado River over the crest of the Austin Dam. Its great turbidity and "soupy" nature, which is at times 15 000 to 20 000 parts per million, strongly supports the writer's idea.

If the floods from more than 30 000 sq. miles of catchment area have resulted in a siltation of only 2 800 acre-ft. per annum (instead of double or thrice this amount), it seems fairly clear that the bulk of the silt brought down by large river discharges must have passed over the dam. It would further seem that, had the impounded water body been of such contour as to constitute a "lake" in the usual concept of such a body, instead of a deepened river channel, its 50 000 acre-ft. capacity might have been 95% silted in 6 to 8 years.

Perhaps a 500 000 acre-ft. reservoir would have been the basis of an economically practicable project at Austin, but the one constructed never was, and never can be, an economic success because of an over-sized water-shed and an under-sized lake. A power company now contemplates constructing two 2 000 000 acre-ft. reservoirs farther up the river. Such reservoirs might help the situation at Austin Lake.

H. F. ROBINSON,* ESQ. (by letter).—In connection with the observations of the author, the status of the silting problem at the Zuni Reservoir in New Mexico is of interest.† The conditions affecting the two cases are quite dissimilar. At Austin, there is a long, narrow reservoir in a perennial stream and, at Zuni, a reservoir almost circular in shape is formed in the bed of an ephemeral stream which is supplied by a very uncertain rainfall and run-off. In both cases large quantities of silt have been brought in and deposited, at Austin, 95.39% of the storage capacity in 13 years, and at Zuni, at the 990-ft. contour, 94.89% in 22 years.

The showing of such immense quantities of silt being brought into reservoirs, under dissimilar conditions, when constructed in stream channels, espe-

* Superv. Engr., U. S. Indian Irrig. Service, Albuquerque, N. Mex.

† See, also, *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 868.

cially in this western country where erosion is increasing from year to year, is not only astounding, but alarming.

From his observations on the Zuni Reservoir, the writer cannot quite agree with the theory of silt deposit advanced by Dean Taylor. Ordinarily, the amount of silt deposited in a reservoir depends on (*a*) the quantity of water coming into, or passing through, the reservoir; and, (*b*) the amount of silt carried by the water. The latter is a variable figure depending on many things, including the character of the drainage area. Under the conditions existing at both reservoirs there is coming into, or through, the reservoir a certain quantity of water each year, which water will carry an unknown but variable amount of silt. It would seem that irrespective of the size of the reservoir (within reasonable limits), with a given quantity of water going into or through it, there would be deposited a similar volume of silt; and as the area of the reservoir was reduced, the deposit of silt would be greater in depth in each period.

Of course, a given quantity of water coming into a large, empty reservoir and remaining there would deposit a greater amount of silt than would the same quantity of water flowing into a reservoir already full, or partly full, and passing over the spillway or dam, carrying a portion of its silt. However, this would be only one minor factor affecting the result.

The "Theory of Silting", as stated by the author, would seem to take none of these factors into consideration, but rather to assume that, knowing the capacity of the reservoir before and after silting and the period elapsing, the rate of deposit is wholly a function of the capacity of the reservoir.

The records of the Zuni Reservoir (Table 4) seem to indicate an irregular deposit which has no reference to the capacity of the reservoir and only indirectly as to (*a*) the quantity of water, with (*b*) the silt content being almost undetermined. The curve of silt deposited annually will be practically a straight line (see Fig. 7*) rather than a curve as shown in the author's Fig. 5.

It is perhaps advisable to give the conditions that have been found at Zuni. After the failure of 1909 the weir was built to Elevation 991, but since then a permanent weir of concrete has been constructed to Elevation 997 and stop-planks can be used to raise it to the original 1 000-ft. elevation.

Silt Deposit.—The quantity of silt brought into the reservoir has been very great. Records are now complete to January 1, 1928, a period of approximately 22 years (Table 4). With the water surface in the reservoir at Contour 990, there is only 5.29% of the capacity left. The amount of silt brought into the reservoir, however, has been greater than this would indicate, the excess being deposited between the 990 and 1 000-ft. contours. The total capacity at Contour 990 was 10 230 acre-ft. The total silt deposit in the reservoir on January 1, 1928, was 11 940 acre-ft. The area of the reservoir at the 990-ft. contour is 512 acres and at the 1 000-ft. contour, 620 acres. The capacity remaining at the 1 000-ft. contour is 26.9% of the original capacity of 15 811 acre-ft. at that height.

* See, also, *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 871, Fig. 1.

The first attempt to determine the amount of silt coming in was in June, 1910, when soundings were taken from a boat, and at intervals thereafter soundings were made through the ice, usually in January. There are only a few weeks in any winter when the ice is thick enough for this purpose and in several years it was impossible at any time in the winter to get out on the ice. This was the case for the several years of no record (Table 4).

In making these soundings the same base line was used each year and the surface of the reservoir was divided into squares 300 ft. on the side. This assured that the soundings for each year were made at practically the same spot and the records, therefore, are unusually close. From these soundings contour plats were made from which the amount of silt deposited was computed. Table 4 shows in detail the silt deposited by periods and the total run-off entering the reservoir for the same periods. The capacity of the reservoir at Contours 990, 1 000, and 1 010 are shown, with the percentages remaining of the original capacity at each of the periods under consideration.

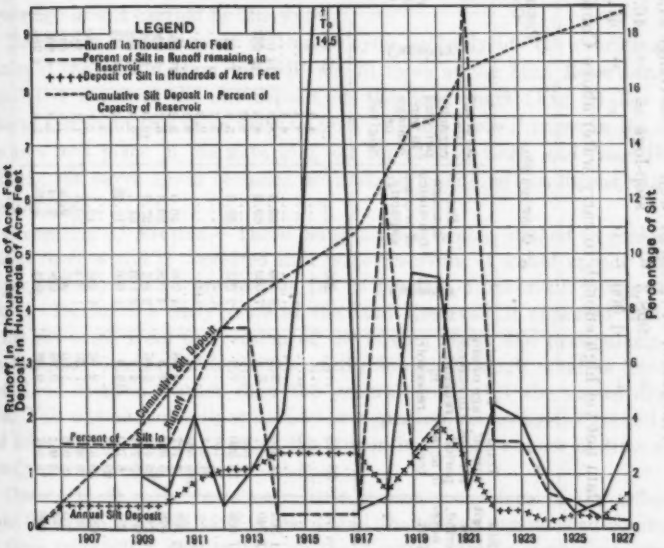


FIG. 7.—SILT PROBLEM OF THE ZUNI RESERVOIR AT THE 990-FOOT CONTOUR.

The reservoir may be considered defined by the 990-ft. contour. Many of the data have been recorded in Fig. 7. In order to compare the various elements, the run-off is expressed in thousands of acre-feet, the deposit in hundreds of acre-feet, and the percentage of silt in the run-off which remained in the reservoir. This may assist in an understanding of conditions and of the original statement* that "the percentage of silt carried by the water is in a certain inverse ratio to the magnitude of the run-off, within, as yet, undetermined points."

* *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 888.

TABLE 4.—SILT DEPOSITS IN ZUNI RESERVOIR.

(Elevation 997, top of weir = Gauge 44.8
" 998.3 " " stop-logs = " 46.1
" 1 000 " " " = " 47.8
Dam not yet high enough to carry water above Contour 1 000.)

Year.	Total silt deposit cumulative, in acre-feet.	Length of period, in years.	Silt deposit for period, in acre-feet.	Total run-off for the period, in acre-feet.	Percentage of silt in run-off remaining in reservoir.	At Contour 990.			Present Maximum at Contour 1 000.			Dam to be Raised to Contour 1 010.		
						Capacity.	Percentage remaining of original capacity.	Average lost capacity per year in percentage of original.	Capacity.	Percentage remaining of original capacity.	Average lost capacity per year in percentage of original.	Capacity.	Percentage remaining of original capacity.	Average lost capacity per year in percentage of original.
1906	0	....	.....	38 000	.....	10 230	....	....	15 811	....	....	20 389	....	....
1907	.....	....	.....	.....	.....	.....	....	4.4	.....	....	2.9	.....	....	2.2
1908	.....	....	.....	18 140*	.....	.....	....	4.4	.....	....	2.9	.....	....	2.2
1909	.....	....	.....	.....	.....	.....	....	4.4	.....	....	2.9	.....	....	2.2
1910	1 804	4	1 804	24 680†	3.2	8 409	82.3	8.8	14 007	88.4	5.7	18 585	91.1	4.4
1911	3 144	1½	1 340	.....	5.4	7 069	69.2	8.8	12 667	80.0	5.6	17 245	84.5	4.4
1912	.....	....	.....	4 080	....	.....	....	5.2	.....	....	3.3	.....	....	2.5
1913	4 200	2	1 056	10 330	7.3	6 013	58.8	5.2	11 611	73.4	3.3	16 189	79.4	2.5
1914	.....	....	.....	21 150	.....	.....	....	3.2	.....	....	1.9	.....	....	1.6
1915	.....	....	.....	54 811	.....	.....	....	3.2	.....	....	1.9	.....	....	1.6
1916	.....	....	.....	146 588	....	.....	....	3.2	.....	....	1.9	.....	....	1.6
1917	5 507	4	1 307	3 777	0.58	4 706	46.0	3.2	10 304	65.8	1.9	14 882	72.9	1.6
1918	6 185	1	678	5 323	12.5	4 028	39.3	6.7	9 626	60.8	5.0	14 204	69.6	3.3
1919	7 433	1	1 248	46 478	2.66	2 801	27.3	12.0	8 292	53.0	7.8	12 956	63.3	6.1
1920	9 296	1	1 863	45 711	4.0	2 503	24.5	2.8	7 308	41.3	11.7	11 120	54.5	9.0
1921	10 589	1	1 291	6 770	19.0	1 666	16.28	8.22	6 114	38.2	3.1	9 829	48.2	6.3
1922	.....	....	.....	22 723	.....	.....	....	2.1	.....	....	1.9	.....	....	1.5
1923	11 271	2	684	20 375	2.96	1 248	12.1	2.1	5 480	34.8	1.9	9 242	45.3	1.5
1924	11 388	1	117	8 865	1.30	1 074	10.5	1.6	5 313	33.6	0.7	9 125	44.4	0.9
1925	11 653	1	265	2 645	1.00	925	9.04	1.5	5 048	31.9	1.7	8 860	43.4	1.0
1926	11 839	1	186	4 981	0.38	728	7.11	1.9	4 862	30.7	1.2	8 674	42.5	0.9
1927	11 940	1	606	16 017	0.38	541	5.29	1.8	4 256	26.9	3.8	8 605	42.2	0.3

* January, 1908, to June, 1910.
† June, 1910, to January 1, 1912.

Two important considerations that were not stated modify that conclusion. For one thing the statement is true only so long as the regimen of the run-off area remains undisturbed. Again, this statement and the data given in Table 4 refer to the percentage of silt in the run-off. This latter is not entirely true, for what is shown is the percentage of silt in the run-off which remains in the reservoir. A certain amount will remain in suspension and pass over the spillway and the part dropped in the reservoir will depend on many conditions, such as the state of the reservoir when a given run-off occurs; whether empty or full; the character of the silt, etc. Hence, the writer is not as sure as formerly that there is any intimate connection between the amount of silt carried by a particular flood and its magnitude. It should also be remembered that a part of the silt does not remain in the reservoir permanently because some is carried out with the irrigation water. It has been estimated that at least 50 acre-ft. goes out in that way annually at the Zuni Reservoir. However, the record is very clear that the greater the run-off the less will be the percentage of silt carried by the water.

The theory of silt deposit of Dean Taylor probably fits the conditions and results at the Austin Dam, but will not fit those at the Zuni Reservoir. The cumulative curve of the silt deposit on the Zuni chart (Fig. 7) lies fairly close to a straight line from 1906 to 1923. At this time changes in the drainage area took place in the nature of silt retardation work, and from 1923 to 1928 the silt curve again becomes, with slowly changing conditions, almost a straight line.

Prevention of Erosion.—There being no practicable method to remove silt once it reaches and is deposited in a large reservoir, it would appear that the proper procedure is to prevent, as far as possible, further silt being carried into it. During the early years of the Zuni Reservoir it appeared that most of the silt came from the erosion of the stream above and particularly from the branches coming in from the south, flowing through what is known as Horsehead Canyon, rather than the general erosion of the entire drainage area. This was undoubtedly caused by over-grazing, removing the ground cover, and allowing the run-off to reach the stream in a shorter space of time and in greatly augmented quantities.

During these early years very little erosion took place in the tributaries from the north. About 1922 it was found that heavy erosion had commenced on these tributaries, particularly on the Nutria. At the same time examinations led to the belief that the erosion on the southern tributaries had about reached an equilibrium and the danger of further heavy deposits from that direction was practically removed.

There are two phases of erosion in the drainage area of streams. One is the lowering of the channel bed and the retrogression of grades through the fine alluvial deposit of the valleys, which continues until bed-rock or hard-pan is reached, the channel widening at the same time in proportion to its increased depth. The other, which is really a part of the same action, occurs where the stream carrying the flood has a sharp bend in which the water striking the outer bank of the curve, breaks off large masses and the eroded

* January, 1909, to June, 1910.
† June, 1910, to January 1, 1912.

material is carried down the stream. Both these conditions are usually found in the same stream.

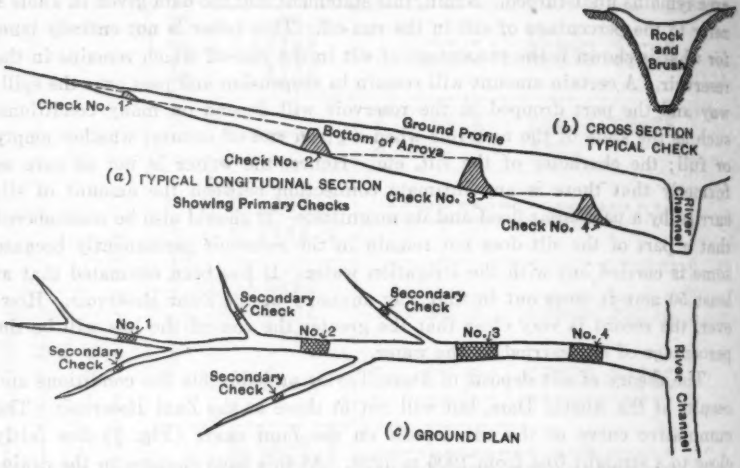


FIG. 8.—DIAGRAMMATIC PLAN OF CHECKING EROSION IN CHANNELS AS APPLIED TO THE ZUNI PROJECT.

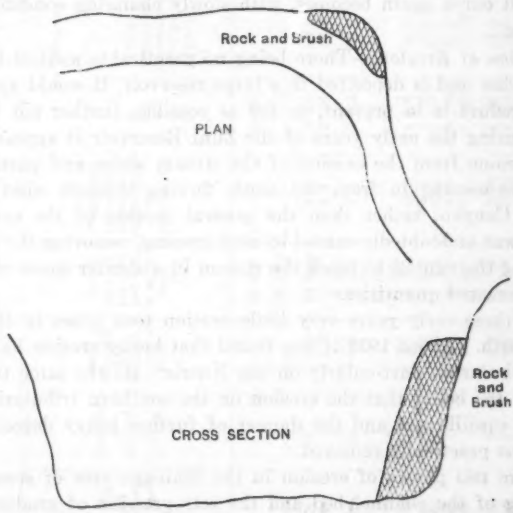


FIG. 9.—CHECKING EROSION ON SHARP TURNS IN STREAM.

It was planned to check this erosion under the first mentioned condition by constructing rock and brush checks which are placed not only in the bottom, but are carried up the banks to a sufficient height to prevent bank erosion. It was decided to place these checks close enough together to form a new grade

so that the velocity of the water will not exceed what it originally was, and the current will not erode the material (Fig. 8).

In the other case, masses of brush and rock were placed at the critical points along the side to prevent erosion. The protection at sharp bends as illustrated in Fig. 9 will not always be effective, for the simple reason that if the bottom of the arroyo is still in soft material the current will erode under the protection work; but in such cases it is only necessary (*a*) to excavate down to harder material; (*b*) to build a protection mass so it will settle with the scouring; or (*c*) to use this side protection in connection with the checks.

The erosion on the north side of the water-shed assumed alarming proportions only in 1921 and 1922, and in 1923 a small sum of money was expended in the protective work outlined. A limited amount of this work has been continued each year since, at the most critical places.

That this has been effective is shown by the fact that the reservoir capacity curve (or what is the same thing, the curve showing the silt deposit (Fig. 10)) is suddenly flattened from 1922 onward. This indicates clearly, unless several consecutive coincidences have occurred, that the protective checks and jetties are doing their work.

An extension of the control work is planned from year to year as money is available and at points indicated as necessary by further erosion.

Processes of Erosion.—As to the general problem of erosion in the Southwest, an office "Watershed Handbook" prepared by the U. S. Forest Service, Southwestern District, contains some valuable comments which may well be repeated in substance as bearing directly on the large question.

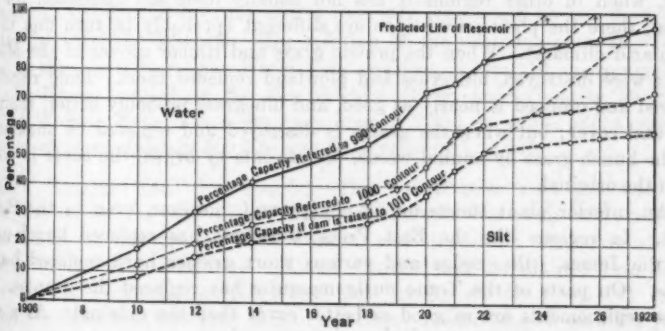


FIG. 10.—SILTING OF ZUNI RESERVOIR. CAPACITIES IN PERCENTAGES OF VOLUMES BELOW GIVEN CONTOURS.

The distinction between normal and abnormal erosion is a frequent cause of misunderstanding. A certain kind and rate of erosion, of course, exist naturally in every place, but are seldom the same in any two places. For any locality they are determined by the interplay of two forces:

1.—Resistance.—Vegetative obstruction, growth, or healing power of vegetation, kind and arrangement of soil, rocks, and débris.

2.—Disintegration.—Kind, amount, and distribution of rainfall, water flow, freezing and thawing, wind, etc.

Whenever, through a long period of time (say, a generation) these two forces approximately balance, and no material change in the configuration of the country occurs, the country may be said to be in equilibrium, and the state of erosion may be called normal. Whenever, through a similar period of time, a material change occurs (such as the disappearance of creek bottoms or gullying of slopes) the state of erosion may be called abnormal. Abnormal erosion occurs locally in the Southwest while the land is still in its virgin condition, but it was exceptional, both as to time and place, because the natural forces which upset the equilibrium (cloudbursts, for instance), were local and temporary. The natural forces for restoring equilibrium (vegetation, for instance), usually had time to "get in" their work before the upsetting process was repeated.

The artificial forces which are now upsetting equilibrium (over-grazing, for instance), are usually not local and temporary, but widespread and of long standing. Hence, the fact that abnormal erosion, once exceptional, is now general. Once self-healing, it is now cumulative. This argument does not assume that there are more or worse cloudbursts now than before the country was settled. It does assume, however, that the same number and kind of cloudbursts have more effect. For practical purposes, no distinction need be made between normal and abnormal. Any erosion that is doing damage and that can be controlled, should be controlled. There is no danger of the effort being so successful as to interfere with normal or beneficial erosion.

Why should settlement upset equilibrium on a wholesale scale in the Southwest, when in other regions it has not usually done so? Because, for one thing, there the plant successions are different (probably in turn due to the semi-arid climate). When the prairie grass and timber groves of the Middle West were destroyed, bluegrass and plowland replaced them. Land regularly plowed and cropped is nearly as good, and bluegrass probably better, than the original cover; but when the grama is destroyed and replaced by snakeweed, or the bunch-grass by annual weeds, or side-oats by brush, the cover is poorer than the original.

An inferior plant succession is not general, however, even in the Southwest. In regions like the East Pecos, bluegrass has replaced bunch-grass. On the Jemez, *valles* sedge and various short grasses have replaced bunch-grass. On parts of the Tonto curly mesquite has replaced the gramas. All these replacements are as good or better cover than the original. As a rule, however, the replacements have been inferior.

Restoration of equilibrium eventually occurs naturally, even where upsetting forces continue at work. The best example is in the region of Santa Fé, where the lower country around the Spanish settlements has been overgrazed for three centuries. The watercourses have simply eroded until they exposed enough rock and cobbles to furnish a mechanical resistance equal to the original vegetative resistance. Thus stabilized, the watercourses are proceeding to re-vegetate with alder, juniper, rabbit-brush, or whatever can grow in the rocks. The new equilibrium, however, is a relatively unproductive one from the economic standpoint.

JULIAN MONTGOMERY,* M. AM. SOC. C. E. (by letter).—Engineers who are concerned with the design and development of storage reservoirs in the southwest section of the United States should greatly appreciate this valuable contribution.

The geology of the drainage area, together with the relation between the size of that area and the capacity of the storage reservoir, are factors that designing engineers must constantly keep before them.

Familiarity with the hydrology of the Southwest reveals that the run-off of the water-sheds of the streams in that section carry considerable silt. It is not unusual for engineers engaged in the design of storage reservoirs to allow from 0.3 to 0.5 acre-ft. of siltage per year per square mile of drainage area.

The relation of the drainage area above Lake Austin, in square miles, to the storage capacity of old Lake Austin, in acre-feet, was approximately 1 to 1.6.

Lake Wichita, the water supply for Wichita Falls, Tex., was built on Holliday Creek about 1900. It has a drainage area of 135 sq. miles and a storage capacity of 14 500 acre-ft. The geology of the water-shed is largely Permian. There is considerable silt in the run-off. Yet flood waters pass over the spillway with sufficient regularity to prevent any particular accumulation of silt. Cross-sections taken in 1925 under the supervision of Dean Taylor show very little siltage in the twenty-five years. The cross-sections also show that scouring has taken place, in general, along the path of the current. At the breaks and curves along the shore line silt has been deposited. Probably the average net deposit over the entire reservoir area would be less than 1 ft. for the life of Lake Wichita. The relation of the drainage area, in square miles, to the capacity of Lake Wichita, in acre-feet, is 1 to 110, approximately.

Lake Kemp, located on the Big Wichita River, approximately 50 miles west of Wichita Falls, has a drainage area of 2 000 sq. miles and a storage capacity of approximately 500 000 acre-ft., a ratio of 1 to 250.

KIRK BRYAN,† ESQ. (by letter).‡—The quantitative data presented by Dean Taylor on the deposition of silt in Austin Reservoir add valuable information on the processes of erosion and deposition characteristic of streams in the arid Southwest. These streams may be divided into three classes:§ (1) Perennial, or those that flow nearly all the time; (2) intermittent, or those that flow only part of the time but more than a month a year; and (3) ephemeral, or those that flow only after rains and less than a month a year.

The perennial streams carry some silt at low-water stages and may be particularly active during the closing part of the flood-water period. Ephemeral and, to a less extent, intermittent streams carry silt only during floods.

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† Asst. Prof. of Physiography, Harvard Univ., Cambridge, Mass.

‡ The data in this discussion formed part of a report to the Middle Rio Grande Conservancy District, September, 1927, on "Erosion and Control of Silt on the Rio Puerco, New Mexico," by Kirk Bryan and George M. Post, published by permission of J. L. Burkholder, M. Am. Soc. C. E., Chief Engineer.

§ Note the excellent definitions of O. E. Meinzer, "Outlines of Ground Water Hydrology," U. S. Geological Survey *Water Supply Paper No. 494*, pp. 57–59, 1923.

Erosion and deposition of silt by such streams are subject to changes and fluctuations quite distinct from the same processes in perennial streams. Thus, the Lower Gila in Arizona and the Rio Grande in New Mexico show a tendency to aggrade their beds, whereas their ephemeral tributaries are actively degrading their channels. This accelerated erosion, or arroyo cutting,* was begun within the memory of men now living. It has vastly increased the total quantity of silt carried by the ephemeral streams. This increased quantity of silt menaces reservoirs constructed to store flood waters. Similarly, in the valleys of the major streams, the abnormal supply of sediment chokes the channels of perennial streams, and induces a number of consequences most of which are troublesome to the works of Man.

The conditions of erosion and sedimentation on the water-sheds of the ephemeral streams have been the subject of much speculation and study with a view of determining the cause of the accelerated erosion or arroyo cutting. A theoretical analysis of the conditions indicates that, because of the increased growth of vegetation, years of large precipitation will be times of minimum erosion by running water and, consequently, times of minimum silt transfer to the lower reaches of the stream. In these periods of larger rainfall, greater quantities of water will flow down the hill slopes through the minor channels, but the erosive power of this water will be much reduced because of the increased vegetative cover. Not only does this principle seem to apply to the short periods of one or two years which frequently occur and are recorded in existing weather records, but many claim that it will also apply to longer periods. Thus, in analyzing the effect of possible fluctuations in climate in past times, this principle has been used. Periods of erosion are thought to be the result of arid climate and periods of alluviation and filling of stream channels are attributed to less arid climate. In regions of greater rainfall and on perennial streams it seems likely that the principle will not hold and the results of a change in climate may be exactly reversed.

Although this principle has been advocated by eminent men and seems sound so far as a mere analysis may go, no definite evidence has heretofore been brought forward. The experience in the operation of the Zuñi Reservoir, however, gives qualitative evidence that this principle is true.

Zuñi Reservoir on the ephemeral Zuñi River in McKinley County, New Mexico, acts as a trap to retain the sand and mud carried by the stream. The area of the water-shed above the reservoir is said to be about 650 sq. miles. Data on the river, its reservoir, and the silt problem have been published by H. F. Robinson,† Supervising Engineer, U. S. Indian Irrigation Service, who has also supplied additional information used herewith.

The silt produced by the Zuñi water-shed is variable in absolute quantity and in relation to the water entering the reservoir. Twelve determinations of

* Bryan, Kirk, "Date of Channel Trenching (Arroyo Cutting) in the Arid Southwest," *Science*, n.s., Vol. 62, pp. 338–344, 1925; "Channel Erosion of the Rio Salado, Socorro County, New Mexico", U. S. Geological Survey, *Bulletin 790*, pp. 17–19, 1927; "Historic Evidence of Changes in the Channel of Rio Puerco, a Tributary of the Rio Grande in New Mexico," *Journal of Geology*, Vol. 36, pp. 265–282, 1928; also, Swift, T. T., "Date of Channel Trenching in the Southwest", *Science*, n.s., Vol. 63, pp. 70–71, 1926; and Wynn, Fred, "The West Fork of Gila River", *Science*, n.s., Vol. 64, pp. 16–17, 1926.

† "Silt Problem of the Zuñi Reservoir," *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), pp. 868–878, and discussion, pp. 879–893.

the quantity of silt deposited in the reservoir have been made, of which seven are for single years, and five for periods ranging from 1½ years to 3½ years. If the quantity of silt for these periods be divided by the mean annual discharge from the reservoir for the same period, as shown by Mr. Robinson in Table 4,* there is obtained an average figure for the silt in percentage by volume carried in the muddy water.

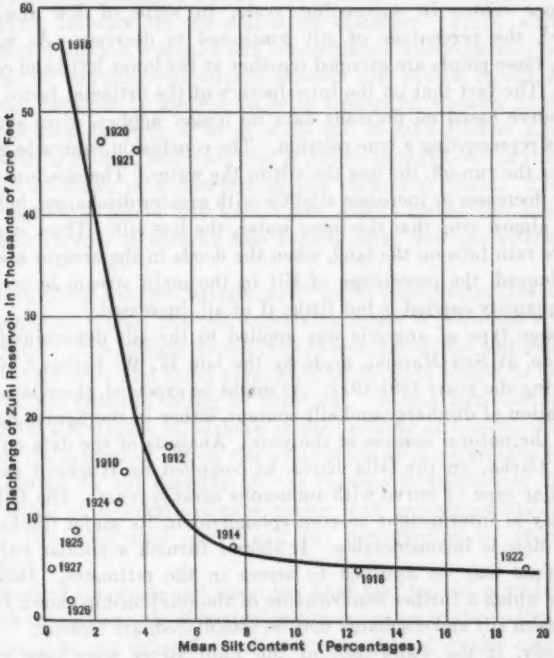

FIG. 11.—RELATION BETWEEN RUN-OFF AND SILT CONTENT.

If these mean discharges are plotted as ordinates and the percentages of silt as abscissas, as shown in Fig. 11, the points representing the periods down to 1924 show a characteristic relation between the run-off from the water-shed and the silt content of the water. The points representing 1923, 1924, 1925, 1926, and 1927, are affected in their position by the silt-control works begun in 1923. The points representing the earlier periods fall fairly close to a smooth line. The discrepancies may be due to errors in determination or to some natural condition. The period ending June, 1910, covers 3½ years, and the quantity of silt should be more accurately determined than in the period of 2½ years ending in January, 1912, represented by the near-by point; but the discharges from the reservoir for 1906 to 1908, inclusive,

* See p. 1698.

were not precisely determined and the estimate may be sufficiently in error to cause a misplacement of the point for the period, 1910. The points representing 1920 and 1921 appear to be in error as a result of miscalculation of the quantity of silt shown by the survey of 1920.

The building of structures to prevent excessive erosion was begun in 1923. On the next measurement of silt in January, 1924, the quantity deposited in Zuñi Reservoir was relatively small and the proportion of silt in the inflow fell to 2.96 per cent. In succeeding years, in spite of low run-off for the water-shed, the percentage of silt continued to decrease. As may be seen in Fig 11, these points are grouped together at the lower left-hand corner of the diagram. The fact that on the introduction of the artificial factor of silt control the curve based on previous data no longer applies, gives support to its validity as representing a true relation. The conclusion cannot be escaped that the greater the run-off, the less the silt in the water. The absolute quantity of silt either decreases or increases slightly with greater discharges, but so slightly that it is almost true that the more water, the less silt. Thus, in rainy years when more rain falls on the land, when the floods in the arroyos are larger and more prolonged, the percentage of silt in the main stream is lower, and the absolute quantity carried is but little, if at all, increased.

The same type of analysis was applied to the silt determinations on the Rio Grande, at San Marcial, made by the late W. W. Follett,* M. Am. Soc. C. E., during the years 1897-1912. As might be expected, there is no similarity in the relation of discharge and silt content, either in the figures for full years or any of the natural seasons of the year. Analysis of the data on silt at and near San Carlos, on the Gila River, as compiled by Hughes,† gave a somewhat similar type of curve with numerous erratic years. The Gila River at this locality is intermittent or even ephemeral in its major discharge, as the low-water flow is inconsiderable. It should furnish a similar curve and the erratic points may be ascribed to errors in the estimates. Data on other streams by which a further confirmation of the relationship shown for the Zuñi River between silt and discharge may be established, are lacking.

Obviously, if the water-shed of the Zuñi River were bare ground, the decrease of silt with run-off would be impossible. Some agent dependent on rainfall must intervene to prevent erosion at times of large run-off. This agent is the vegetative cover which is much reduced in effectiveness in dry years. Hence, the more rain, the more grass, and, consequently, the less erosion and the less silt carried in the streams.

Students of forestry and grazing have long contended that artificial reduction of the vegetation by over-grazing promotes erosion and increases the silt content of streams. Many writers, following this line of reasoning, have held that the destruction of grass cover by over-grazing is the cause which induced the cutting of existing arroyos. On the other hand, others hold that a change in climate by which the rainfall is reduced, will produce the same result.

* Follett, W. W., "Silt in the Rio Grande"; Commission on Equitable Distribution of Waters from the Rio Grande, Special Rept. to the Secy. of State, State Dept., 1913.

† Hughes, D. E., "Amount of Silt That Would Be Deposited in a Reservoir at San Carlos," etc., H. R. Doc. 791, 63d Cong., 2d Sess., pp. 117–140, 1914.

The data in Fig. 11 represent merely the relation of silt to rainfall. They do not specify where the silt will originate. Obviously, vegetation is most effective in decreasing erosion of hill slopes, on alluvial fans, and at the head of gullies. After the water is concentrated in the valley floors, a good growth of grass tends to spread this water and keep it spread, by which means silt is brought down more slowly. A decrease in vegetation, particularly grass, whether brought about by over-grazing or a change in climate to greater aridity, induces the formation of channels. Thus, although the general erosion decreases, a special and concentrated stream erosion may ensue.

This type of concentrated stream erosion has within recent years produced channel trenches or arroyos on most ephemeral streams of the Southwest. The cutting of arroyos, or as some call it—the Recent Epi-Cycle of erosion—seems to be due, following the principle brought out in the measurements of silt at Zuñi, to a decrease in the vegetative cover. The delicate balance of erosional forces has been upset and, whereas the streams once aggraded their valleys, or were at least neutral, they now are actively degrading them.

That over-grazing was the "trigger pull" which precipitated this accelerated erosion is apparently true, but the underlying cause is doubtless a change to a dryer climate. This change probably occurred before the settlement of the country by white men, and there appears to be no good reason to believe that a reversal of the climatic swing is imminent. Sound engineering practice dictates careful silt studies in advance of the building of reservoirs and active measures for the control of silt on the water-sheds of those already built. The successful control of silt depends on (1) reasonable restriction of grazing; (2) introduction of drought-resistant plants that are non-palatable to stock (salt cedar (*Tamarix* sp.,) etc.); and (3) construction of retarding and retaining works.

The success of the works put in on the Zuñi may lead to a tendency to put faith in such direct methods of control to the exclusion of the indirect methods which can only be put into effect after a campaign of public education and with the aid of State authority.

J. C. Stevens,* M. Am. Soc. C. E. (by letter).—The author has brought forcibly to the attention of the profession and to the world the seriousness of the silt problem in one of its most menacing forms. The loss of storage capacity, amounting in effect to nullifying large investments in storage works, is a problem the seriousness of which is realized in only a few quarters.

What is to become of the civilizations that will grow up as a result of the creation of storage works on the silt-laden streams of the West? Are lands now teeming with life and verdure doomed to revert to sagebrush and cactus? Is science powerless to halt the march of this relentless foe?

It is staggering to think that in so brief a period as 7 years the capacity of a 48 000 acre-ft. storage reservoir was reduced to one-half and that in a subsequent period of 13 years its usefulness has been practically obliterated.

* Cons. Hydr. Engr. (Stevens & Koon), Portland, Ore.

P. A. WELTY,* M. AM. SOC. C. E. (by letter).—This paper gives testimony as to the seriousness of the problem of silt deposit in the canals and reservoirs of the Southwest. All streams of this region do not carry such excessive quantities of silt, but most of them do, and where this condition does obtain, it seriously affects the operation of canal systems and the life of the reservoir. There are a few reservoirs already constructed in Texas, and others contemplated, which will not be reduced in storage capacity appreciably in several centuries, and there are others built and contemplated that will be greatly reduced in capacity in an average man's life time.

During two periods, 1907 to 1909 and 1916 to 1918, while engaged as an engineer, in the Lower Rio Grande Valley of Texas, on the San Benito Project, the writer had the opportunity to observe the effects of silt on canal operation. The Lower Rio Grande Valley is a delta country that has been built up from the levels of the sea, or the Gulf of Mexico, by the deposit of silt. The river itself flows along a ridge, or backbone, which is (or was until very recently) being constantly raised in elevation along its course, by silt deposited when the river overflows its banks, and also by blow-sand. When these over-flows occur, the water leaves the channel and flows across country at right angles, approximately, to the stream. It runs into Mexico on the right side and across the delta plain toward the Gulf of Mexico on the left side in the State of Texas. As these over-flows occur without much velocity, the waters deposit their heavy loads of silt quickly. The heavier grains are dropped first, and the smaller ones next in order, and so on, some of the finer particles being carried until low depressions are reached.

There are many old former river beds of the Rio Grande (locally called "resacas") meandering through the valley, that have been built up in this manner during the past ages, until the time the river reached a point where it broke away from its channel and sought a new flow line through the lower country which, in turn, it began to raise as before. Some of these "resacas" are mentioned in United States history as being the battlegrounds during the Mexican War in 1848.

More than 500 000 acres of land are now being watered in the lower valley of the Rio Grande, in varying areas, by pumping water from the river, making lifts from 12 to 75 ft. to lands above and conducting it to lands embraced in a score or more of projects down the valley as far as Brownsville, Tex., at the lower end of the river. The plans for the San Benito Project, in Cameron County, included low-lift and high-lift pumps, and, in addition, proposed to obtain gravity-flow water for a large percentage of its lands, by taking water directly from the river. The plans were developed and the works constructed by the Chief Engineer, Mr. Sam A. Robertson.

Referring to Figs. 12 and 13, an idea of project locations can be obtained. Fig. 13(*a*) is a sketch (not to scale) of the profile of the 2-mile dredged channel conveying the waters of the river to the old "resaca" that was to become the Main Canal of the San Benito System. Fig. 14(*b*) shows the typical cross-section of the "resaca", which varied greatly in widths, between banks, from

* Dist. Engr., Bexar-Medina-Atascosa County Water Impvt. Dist. No. 1; Chf. Engr. and Gen. Mgr., San Antonio Suburban Irrigated Farms, Natalia, Tex.

250 to 1 000 ft. and in depths from 15 to 20 ft. The inside slopes of the "resaca" were from 3 on 1 to 5 on 1. The present river through the valley is more uniform and varies but little. The banks are almost vertical for at least 15 ft. from the top.

Fig. 14(*b*) also shows the conditions of the upper few miles after ten years of operation, in the quantity of silt that has been placed in the "resaca", with no thought of maintenance work for its removal.

Fig. 14(*c*) shows a sketch profile (of no regular scale) of the "resaca" and the different stages of silt deposit as it progressively built in down stream.

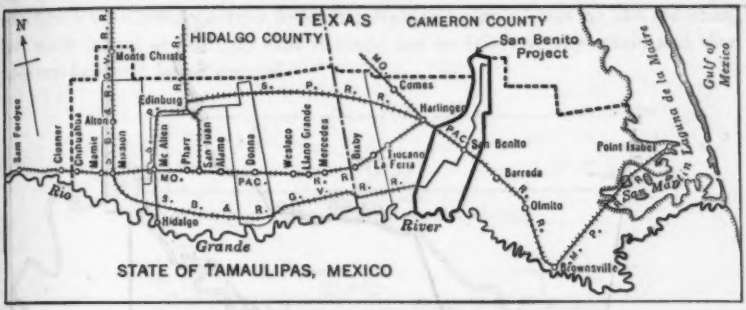

FIG. 12.—GENERAL PLAN, SAN BENITO PROJECT.

At the point marked *G* (Fig. 13), heavy service gates were installed to receive and control the waters of the river. These were operated with screw-lifts and were set in position in massive reinforced concrete walls, carried on both round and Wakefield sheet-piling. The floor of the structure was below the bed of the Rio Grande. The pumping plants were also housed in an adjoining room of this same structure, the suction pipes protruding through the river-front wall.

The profile of the dredged channel beginning at *G* (Fig. 14(*a*)), indicates a cut of 19 ft. at the river, as well as one of 17 ft. at the old "resaca", and one of 9 ft. at a point midway between the two. The finished grade of this channel was 1 in 10 000. The channel dredged along this line was 20 ft. wide at the bottom, with side slopes of 1 on 1.

While the land on each side sloped away from the "resaca", the banks of the latter and its bottom also had a uniform slope continually going parallel to the country, but because of its windings its grade was flatter than a line across country. On the San Benito Project the elevation of the plant shown at *G* (Fig. 13), was about 48 ft. above sea level, and the Town of San Benito was 32 ft.; the profile then smoothed out to sea level at the Gulf. The land had an average slope of 18 in. to the mile.

At intervals along the "resaca", and across it, an earthen dam with a roadway on top was constructed and in the dam were placed locks through which to pass barges and, also, gates with which to control the water. The locks were of reinforced concrete. Below each dam, and as the grade fell, a levee was constructed on each bank with a level grade line, starting with a zero fill and ending at the next dam with a 6 or 7-ft. fill. This plan resulted in the waters

of the Main Canal, or "resaca", being in lifts, or levels, each lower section being about 6 ft. lower than the section above. This water, being as a rule higher than the lands adjacent to the "resaca", made the irrigation of the lands easily possible. Laterals were constructed at every ½ mile, or 1 mile, along the "resaca", at right angles, approximately. These served to water lands on either side in the 40-acre units as subdivided. Some lands along the "resaca" were too high to be thus watered, as were the lands adjacent to the river, and high-line canals were located and built along the "resaca" and the river to care for such areas.

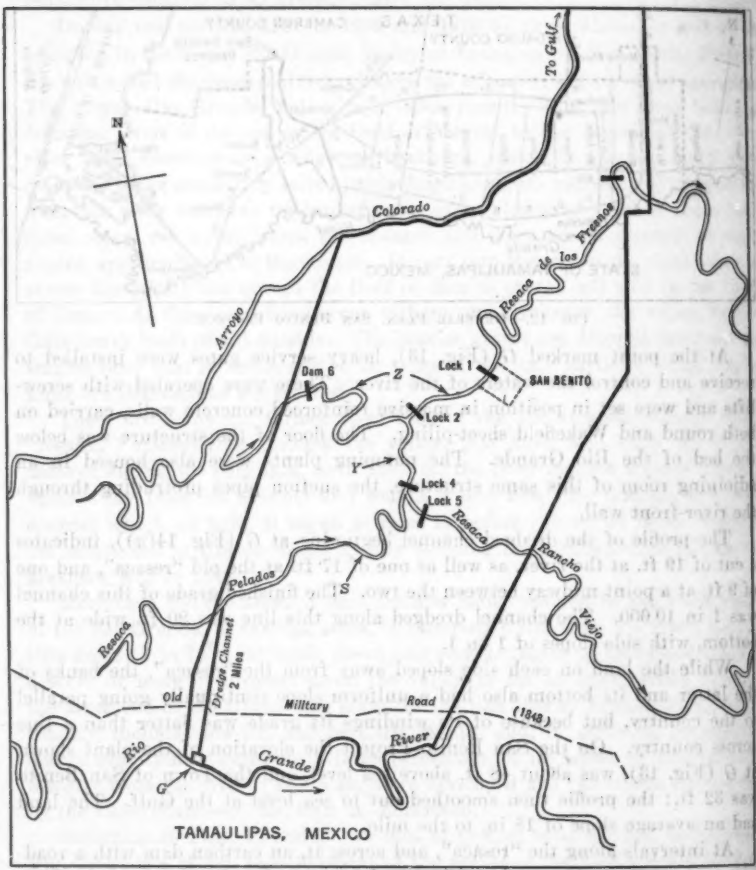

FIG. 13.—OUTLINE MAP, CAMERON COUNTY WATER IMPROVEMENT DISTRICT.

In constructing the 2-mile dredged channel it was necessary to take material from borrow-pits to build up the levees near the middle, but at each end there was excessive waste. This resulted in a channel more than 20 ft.

deep. The failure to keep up maintenance work for the removal of this silt soon resulted in this 2-mile channel becoming so full that water could be taken by gravity only when the river was "up" and, later, only when the river was at flood stage. Subsequently, it became necessary to begin the work of raising banks and making what was originally intended to be a low-line canal designed for taking gravity water, into a high-line pumping canal. This was the status when the writer came to the newly organized District on his second engagement there. This old low-line canal was further raised and strengthened so as to form a new high-line canal, and a new low-line canal was excavated through low country, thereby following Nature's plan of leaving the old channel when it had so built up that it could not be followed any more, and then seeking new and lower ground.

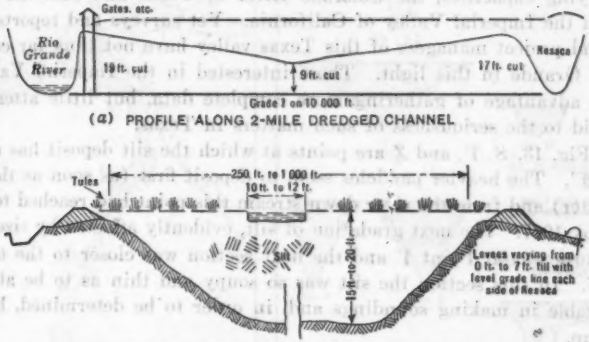

FIG. 14.—PROFILE AND TYPICAL SECTIONS.

The old "resaca" with dimensions as shown in Fig. 14 stored an immense volume of water, and while it was wasteful as regards seepage and evaporation, and was not economical because of the pumping required to fill up the bottom to a height that was usable, yet it was a great land seller and the sight of this great area of water was something that the average prospective buyer could not resist. Consequently, the project grew rapidly until it embraced about 80 000 acres of land, extending down stream along the old "resacas". This steady growth of the project down stream was responsible for the neglect in the upper sections. During the time of the so-called Border trouble, when it

seemed as if the Federal Government might intervene in Mexico, there was a period of neglect in the maintenance work caused by re-organizing into the District form of ownership and also bad financing. By 1916, no water was transferred by gravity and pumping was done only at a great additional expense for installing many re-lift pumps at the laterals. Bank-raising was continually in progress, and in order to meet the changing conditions, new pumps of greater capacity were installed at the head-gates.

For the first ten years of operating the system, the quantity of silt deposited in the 2-mile channel and in the first few miles of the upper end of the "resaca", as shown by measurements made by the engineers in February, 1917, reached a total of something less than 5 000 000 cu. yd. The results of the surveys would indicate that the Rio Grande water approaches, in silt-carrying capacities, the Colorado River of Texas and also the Colorado River in the Imperial Valley of California. Yet surveys and reports of engineers and project managers of this Texas valley have not thus far considered the Rio Grande in this light. Those interested in the Imperial Valley have had the advantage of gathering more complete data, but little attention has been paid to the seriousness of such matters in Texas.

In Fig. 13, *S*, *Y*, and *Z* are points at which the silt deposit has a decided "drop-off". The heavier particles seem to deposit first (as soon as they strike dead water) and from the river down stream this point had reached to *S* (Fig. 14(*c*)) by 1917. The next gradation of silt, evidently affected by size or some other cause was at Point *Y* and the next section was closer to the town at a point, *Z*. In this section the silt was so soupy and thin as to be almost unrecognizable in making soundings and, in order to be determined, had to be dipped up.

Below Point *Z* nothing was indicated and the elevations on land lines taken ten years previously checked measurements to the bottom. These sections of silt deposit seem to be almost level and the "drop-offs" at *S* and *Y* are rather abrupt, the change of elevation seeming to occur in ¼-mile intervals along the canal.

The upper reaches of the "resaca" had become silted to the top of the banks. In some cases the banks had to be raised, except along the channel where the water supply flowed, as indicated in Fig. 14(*b*). This channel was about 12 ft. wide and 2 ft. deep, and meandered through the tules and the silt bed. Laborers were constantly engaged in cutting tules, raising banks, and endeavoring to enlarge and keep open this small channel through which the water supply flowed. The velocity was a little better than 2 ft. per sec., which helped to keep moving the silt that was in suspension, and to carry it forward to the points, *S* and *Y*. This silted section of the "resaca" was under water when the pumps were operating and the tules caused the formation of pools of still water in which the silt would drop immediately. Consequently, this silt bed was continually being raised, which, in turn, made it necessary to keep raising the banks.

Some of the projects of the valley used old "esteros", or lakes, that functioned as settling basins. Levees were built around these lakes and the water

was discharged into them directly from the pumps, only to be admitted to the canal system after much of the silt had settled out. It was then continually necessary to remove the silt from these basins, and raise the banks.

BANKS MCLAURIN,* ESQ. (by letter).—The writer has been associated with Dean Taylor on one of the silt surveys of Lake Austin, and has lived on the banks of the Colorado River (Texas) for a life time. He is, therefore, rather familiar with the river, its topography, floods, droughts, and silt-bearing capacity. The Austin Lake stretches its crooked course from Austin into the mountains to the northwest. Those unfamiliar with the lake would conclude

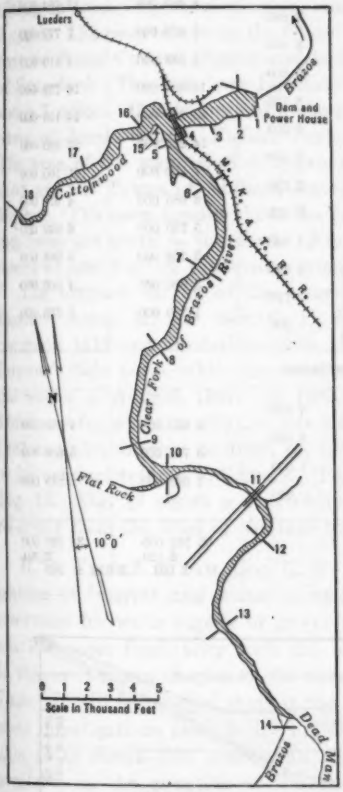

FIG. 15.

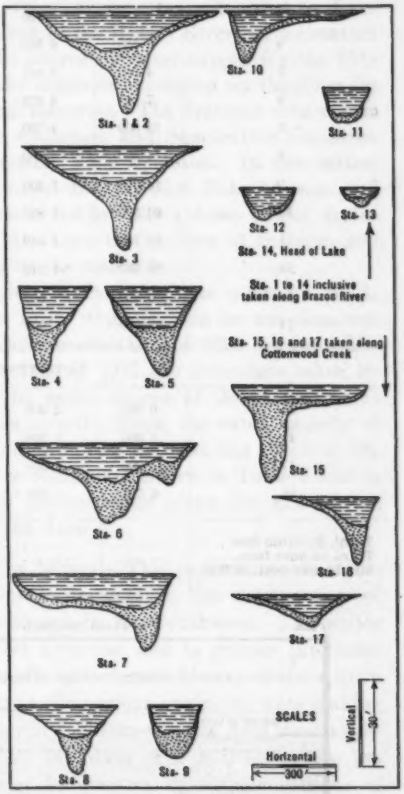

FIG. 16.

that it is a channel reservoir, but this is only partly true. It cannot be classed as a channel reservoir. Fully 50% of its area was formerly on the banks or above the top of the banks. This lake brings into strong contrast the behavior or silting of its channel part and the valley or part that covered former farm lands. Measurements of Lake Austin upset many theories and show con-

* Adjunct Prof. of Civ. Eng., Univ. of Texas, Austin, Tex.

TABLE 5.—VOLUME OF WATER IN LAKE PENICK.

Section No.	Station distance, in feet.	Water Areas, in Square Feet.		Water Volumes, in Cubic Feet.	
		1927	1920	1927	1920
CLEAR FORK OF BRAZOS RIVER.					
1	00	5 500	8 000		
				2 750 000	4 000 000
2	500	5 500	8 000		
				8 480 000	12 704 000
3	2 100	5 100	7 880		
				5 376 000	7 779 000
4	3 500	2 580	3 100		
				3 358 000	4 510 000
5	4 600	3 525	5 200		
				10 500 000	16 719 000
6	7 100	4 875	8 175		
				15 320 000	24 120 000
7	10 300	4 700	6 900		
				14 200 000	20 800 000
8	14 300	2 400	3 300		
				7 800 000	11 100 000
9	18 300	1 500	2 250		
				3 685 000	4 978 000
10	20 500	1 850	2 275		
				5 760 000	6 849 000
11	24 100	1 350	1 530		
				3 108 000	3 564 000
12	26 500	1 240	1 440		
				3 680 000	4 300 000
13	30 500	600	710		
				1 350 000	1 598 000
14	35 000	000	000		
COTTONWOOD CREEK.					
15	0 000	2 000	4 600		
				2 022 000	4 260 000
16	1 200	1 370	2 500		
				3 738 000	5 600 000
17	4 000	1 300	1 500		
				1 625 000	1 875 000
18	6 500	000	000		
Total, in cubic feet.....	.....	.....	.....	92 752 000	134 768 000
Total, in acre-feet.....	.....	.....	.....	2 129	3094
Silt, in acre-feet, 1920-27..	.....	.....	.....	3 094 — 2 129 = 965	

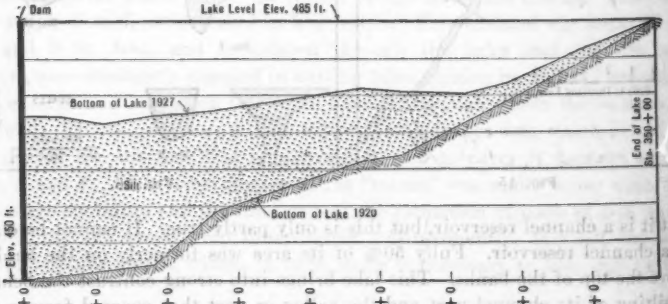

FIG. 17.—PROFILE OF LAKE BOTTOM.

clusively that the valley part of the lake was silted much more rapidly than the channel part. In many places along the 25 miles of lake shore, the silting of the valley part (not in the old river bed) has been more than 100%, and is being increased with every flood. At first, the low-lying banks were only a few feet high, but after each flood, grass, weeds, or willows spring up, and these growths catch trash, drift, etc., retard the velocity of the water, and cause the silt to drop out on the new banks. Slowly but surely the channel width is being narrowed.

Silting of Lake Penick.—Lake Penick, at Lueders, Tex., presents a case of silting in some respects similar to, and, in other respects, in contrast to, Lake Austin. The reservoir is on the Clear Fork of the Brazos River, in the eastern part of Jones County, Texas, and is the source of water supply for the City of Stamford. The dam is at Lueders, the reservoir extending up the river for about 7 miles. This is more of a channel reservoir. The drainage area covers parts of Jones, Scurry, Fisher, Taylor, Callahan, and Shackelford Counties. The area of the water-shed is approximately 2 250 sq. miles. In this water-shed are the Towns of Abilene, Sweetwater, Buffalo Gap, Roby, Anson, and Merkel. The main stream above the dam is fed by fully a dozen creeks, draining from the south to the north. Fig. 15 shows the location of the dam and reservoir and Fig. 16, the cross-sections on the reservoir.

The original survey of the reservoir and dam site was made by E. A. Burrow, Assoc. M. Am. Soc. C. E., in 1918. Construction on the dam was begun in 1918 and finished in 1920. The cross-sections of 1920 are from Mr. Burrow's field notes, while the cross-sections of 1927 are from data taken by the writer in August, 1927. In 1920, the water volume of the reservoir was 3 094 acre-ft., while, in 1927, it was 2 129 acre-ft. Thus, the water capacity of 1927 is 68.2% of that in 1920, or, in other words, the lake has silted 31.2% of its original volume in 7 years. These results are shown in Table 5 and in Fig. 16. Fig. 17 shows a longitudinal section taken along the axis of the reservoir from the head of the lake to the dam.

R. G. Tyler,* M. Am. Soc. C. E. (by letter).—This paper contains information of interest and value to engineers engaged in the construction of reservoirs for water supply or power purposes in the Southwest. The writer has had some familiarity with the local situation and is greatly interested in this most recent chapter in the unusually unfortunate history of the Austin Dam. It will be recalled that, at the time the various engineers were making their investigations prior to the building of the dam, too few data were available as to stream-flow conditions. Very few data of a reliable nature are available on the quantity of silt carried by streams in various sections of the United States. The Federal Department of Agriculture could render a most commendable service to the Engineering Profession by conducting extended investigations on this problem.

From such data as the writer has at hand, it would appear that one might expect this stream to carry a burden of about 1% of its volume, or about 18 000 acre-ft., of silt per year.† It has been estimated that the Brazos River

* Prof of San. Eng., Mass. Inst. Tech., Cambridge, Mass.

† *Bulletin No. 1430*, U. S. Department of Agriculture, p. 24.

above Waco, Tex., with a drainage area of 30 000 sq. miles, carries 3 200 000 tons of soil per year.* These streams are believed by the writer to be somewhat similar, but these estimates give the Colorado nine times the silt burden of the Brazos. The estimate for the Colorado is probably the more accurate as indicated by an analysis of the data given by Dean Taylor.

Table 6 gives the data from *Bulletin No. 1430,* of the U. S. Department of Agriculture, expressed in convenient units. It will be noted that there is an extremely wide variation in the amount of silt carried by streams in different sections of this country. The streams of the northeastern section of the United States are relatively free from silt, due largely to the nature of the tributary area.

The character of the drainage area is an important factor with reference to its silt production. With similar soil conditions, open, cultivated tracts erode more than wooded or grazing areas. In arid areas, also, where high temperatures and low rainfall discourage the growth of vegetation, the turf provides inadequate protection against erosion. These conditions prevail over a large part of the Colorado River drainage area above Austin, and are largely responsible for its high silt burden. This is a condition which has a greater effect as time goes on, due to the increased acreage coming under cultivation. Such erosion is objectionable both from the standpoint of impoverishing the denuded areas and of silting streams and reservoirs. The conservation of forests and the reforestation of denuded areas should have a beneficial effect in decreasing siltation difficulties.

TABLE 6.—SILT CONTENT OF VARIOUS RIVERS.

Stream.	Tons per year.	Drainage area, in square miles.	Tons per square mile per year.
Brazos River, Texas	3 200 000	30 000	107
Missouri River, above Ruegg, St. Louis County, Mo.	176 000 000	528 700	333
Arkansas River, above Little Rock, Ark.	40 000 000	148 000	270
Colorado River, above Austin, Tex.	32 500 000*	34 200	950
Tennessee River	11 000 000	35 000	314
Yadkin River, above Salisbury, N. C.	850 lb. soil per year per acre.	.....	272
Mississippi River, above Minneapolis, Minn.	117 000	19 585	6
Susquehanna River, at Danville, Pa.	240 150	9 530	25
Hudson River	240 000	13 366	18

* Estimated as 18 000 acre-ft.; 50% voids assumed.

Assuming that a silt burden of 18 000 acre-ft. per year is a fair value for the Colorado River, it is evident from the measurements given by the author that the quantity deposited is much less than this. For example, the average deposition per year between 1893 and 1900 was about 3 366 acre-ft., while the first year of this period may have yielded about 7 800 acre-ft., if a distribution similar to Fig. 5 is assumed. During the period, 1913 to 1922, the average deposition was 2 962 acre-ft. per year; while, during 1922 to 1924, it was 1 230 acre-ft. annually, with only 712 acre-ft. per year between 1924 and 1926. Again (see Fig. 5), the maximum silt deposit occurring in 1913 was about 6 400 acre-ft. Thus, an ever greater proportion of silt is carried over

* *Bulletin No. 1430,* U. S. Department of Agriculture, p. 24.

the dam each year, due to the decrease in the period of detention in the lake and to the corresponding increase in the velocity of water flowing through. The shape of the reservoir with its relatively small transverse section is such as to make the silting less than would occur in the more usual type, with greater lateral dimensions. This shape affects the velocity, which effect must be considered in predicting the amount of siltation.

The topography at the location of the dam has been described in various reports and papers, but there is one feature that is worthy of further comment. The valley down stream from the dam gradually widens and considerable quantities of coarse sand and gravel, comparatively free from silt, have been deposited in its upper reaches, from which material has long been taken for concrete aggregate for local construction work. This area extends to a point opposite or below the city proper and, as the valley continues to widen, the sand becomes finer and contains greater quantities of silt, so that a few miles farther down stream there is a broad valley where the soil consists of a sandy loam used for the growing of garden produce. This indicates the character of sediment carried by the stream and deposited at the point where the channel emerges from the canyon section into the valley section and the gradient flattens out from 4.2 to 1.7 ft. per mile.* An examination of this valley would give the engineer a very good idea of the type of material which the stream might be expected to bring down at flood stage. The paper indicates a somewhat similar arrangement of materials in the lake, with its sand deposits for a distance of about 3 miles above Station 17.75 and grading off into silt below that point.

In constructing reservoirs in semi-arid regions, the engineer is confronted with the problem of providing a volume sufficient to allow for silting and still give a reasonable life that, at the same time, provides a large surface area from which excessive evaporation losses occur. The writer has found cases where, with the low rainfall and percentages of run-off and the high evaporation from water surfaces encountered in the Southwest, it has been difficult to find reservoir sites which would supply a reasonable demand. The difficulty is more acute where the valleys are shallow and the reservoir volume is distributed horizontally rather than vertically. In such reservoirs there is considerable shoal water which adds very little to the available storage, but from which a depth of water of 50 to 70 in. is lost in evaporation annually. Large volume, therefore, is not necessarily a solution for the silt problem, as it introduces another problem quite as troublesome.

In some instances, especially abroad, siltation has been controlled by the construction of an upper reservoir with a by-pass around the lower or main reservoir. The water carrying the higher turbidity is by-passed and only the partly settled water is carried into the lower reservoir. A by-pass would have been out of question in the present instance, however, because of the topography.

E. C. H. BANTEL,† M. AM. SOC. C. E. (by letter).—The State Board of Water Engineers, which has supervision over the flowing waters of the

* *Water Supply Paper No. 44*, U. S. Geological Survey.

† Prof. of Civ. Eng. and Asst. Dean, Coll. of Eng., Univ. of Texas, Austin, Tex.

TABLE 7.—QUANTITY OF SILT AND RATE OF SILTING IN RESERVOIRS IN TEXAS AND NEW MEXICO.

State.	Authority.	Reservoir.	Drainage area, in square miles.	Original capacity, in acre-feet.	Period.		Number of years in period.	Silt accumulated, in acre-feet in period.	Silt accumulated, in acre-feet per year.	Silt accumulated per square mile of water-shed.
					From.	To.				
Texas	T. U. Taylor	Old Lake Austin	88 000	49 300	1893.3	1900.1	6.8	23 560	34 600	0.091
	T. U. Taylor	New Lake Austin	88 000	32 029	1918	1922	9	26 667	2 963	0.078
					1922	1924	2	2 461	1 230	0.032
					1924	1926	2	1 424	712	0.019
	Charles Schultz	McKinney	1.56	12	.....	.....	10	12	1.2	0.770
	Banks McLaurin	Lake Penick, Stamford	2 250	3 094	1920	1927	7	965	138	0.061
	J. B. Hawley	Lake Worth	1 800	.....	.....	.....	.....	.....	.....	0.500
New Mexico	L. M. Lawson	Elephant Butte	30 000	2 638 000	1916.8	1925.6	8.8	177 740	20 200	0.670
	H. F. Robinson	Zuni	650	10 230	1906.5	1910.5	4.00	1 804	451	0.690
					1910.5	1912	1.50	1 340	894	1.38
					1912	1914	2.00	1 056	528	0.810
					1914	1918	4.00	1 307	327	50.
					1918	1919	1.00	678	678	1.04
					1919	1920	1.00	1 248	1 248	1.92
	U. S. Reclamation Service	McMillan	22 000	29 000	.....	.....	10.00	12 200	1 220	0.055

State of Texas, has declared that all the ordinary flow of all the rivers and streams of the State has been allotted. Permits have been issued to individuals, corporations, cities, and towns, authorizing them to take from the rivers and streams quantities of water which, in the aggregate, equal their total ordinary flow. Additional permits cannot be granted. The supply is exhausted. Only the storm waters remain unallotted. These are regarded as the property of the State to be disposed of as may be decided by law.

It is also true that at times the ordinary flow of the streams is insufficient to supply present needs. The irrigation companies are constantly clamoring for more water, and many cities and towns have suffered long and severe shortages. Abilene, Dallas, Dalhart, and Elgin may be mentioned in this connection. The population of the State and its agricultural and manufacturing interests are growing steadily and the demand for water is growing proportionately. If an adequate supply is not provided, growth in population and industries will soon cease.

In a large section of the State artesian water is available and will be developed as needed, but in a much larger part there is none. Conservation and use of the now unappropriated flood waters is the only recourse left to this region. Conservation of flood waters implies the construction of reservoirs, and experience shows that these become filled with silt and are useless in comparatively short times. Table 7 shows the quantity of silt and the rate of silting in some reservoirs in the Texas and New Mexico sections.

The silt accumulation, in acre-feet per square mile of drainage area per year, is given in each case. The quantity for Lake Austin varies from 0.091 to 0.019 acre-ft. The average quantity for the 20 years is about 0.07 acre-ft. Compared with the figures for the other reservoirs listed in Table 7, this quantity is small, but the new Lake Austin has been practically filled with silt in fifteen years. During the years when the capacity of the reservoir was great, the quantity of silt stored was also great. As the capacity of the reservoir became reduced by silt accumulation, affording less opportunity for sedimentation, the quantity deposited per square mile of drainage area steadily decreased. This seems natural and logical, but there is no such decrease in the silting of the Zuni Reservoir. Here, the rate of accumulation was greatest when 70% of the original capacity of the reservoir was filled with silt. It is interesting, also, to note that the rates of accumulation in the Elephant Butte and Zuni Reservoirs are much greater than in Lake Austin. The rate for Elephant Butte Reservoir is seven times, and that for Zuni Reservoir varies from seven to twenty times, the maximum rate for Lake Austin. McKinney Reservoir has a high rate of accumulation, 0.77 acre-ft. per sq. mile of drainage area per year.

The method of preventing silting that has shown promise of success is to build dams with under-sluices through which the silt-laden flood waters may be passed. This method is said to be successful in India and Egypt. It may be that it can be adapted to Texas conditions.

R. F. WALTER,* M. AM. SOC. C. E. (by letter).—The original storage capacity of Lake Austin (49 300 acre-ft.), never represented a very appreciable

* Chf. Engr., Bureau of Reclamation, U. S. Dept. of the Interior, Denver, Colo.

proportion of the total annual flow of the Colorado River at Austin, Tex. In fact, it was only 1.4% of the average annual flow of 3 500 000 acre-ft. determined by the U. S. Geological Survey for the period from 1898 to 1922.* With so small a ratio of original storage capacity to total annual run-off on a silt-bearing stream like the Colorado River of Texas it is surprising that the original capacity was not even more rapidly depleted by silt accumulation. However, such a reservoir could hardly be expected to operate efficiently as a de-silting agency and undoubtedly large quantities of silt have been carried over the dam during flood periods. It is hoped that Dean Taylor can supply some information regarding this phase of the problem; also, regarding the maximum rates of flood flow and the velocities at which the floods passed through the lake.

The problem of storage depletion by silt accumulation is one that has often been considered by the U. S. Bureau of Reclamation, especially in connection with the investigation of irrigation projects located in the southwestern parts of the United States where the climatic, topographic, and geologic conditions are conducive to rapid rates of soil erosion and correspondingly high percentages of silt content in river flow. Percentage measurements of silt content, by sampling river flow, have been made at several stations on the Colorado River and its tributaries above Yuma, Ariz., on the Rio Grande and its tributaries, and on other silt-bearing streams of the Southwest. Actual quantitative measurements of silt accumulated in reservoirs have been made at the Roosevelt, Elephant Butte, and McMillan Reservoirs on the Salt River, Rio Grande, and Carlsbad Projects, respectively. This discussion will present the results of the measurements at the Elephant Butte and McMillan Reservoirs. The operation and maintenance of the Salt River Project was assumed by the Salt River Valley Water Users' Association in November, 1917, and subsequent silt surveys of the Roosevelt Reservoir have been made by that organization.

McMillan Reservoir.—Lake McMillan is located in the Pecos River Valley of Southeastern New Mexico, about fifteen miles from the Town of Carlsbad. The Pecos River rises in the north-central part of New Mexico, on the east side of the Truchas Range of the Sangre De Cristo Mountains. For some distance it is a typical mountain stream, flowing with a swift current through rocky canyons and carrying only such materials as may be eroded and brought in by mountain torrents. These materials settle out rapidly and are seldom transported appreciable distances after they reach the main channel. However, farther down stream, where the course of the river is through alluvial soil, more material is eroded. Below Roswell, tributary streams, draining large areas of arid, mountainous country lying west of the Pecos, bring in heavy loads of silt, so that by the time McMillan Reservoir is reached, the flow is heavily charged. The mean annual flow of the river at Lake McMillan is approximately 316 000 acre-ft., the yearly values varying from a minimum of 130 000 ft. to a maximum of 900 000 acre-ft.

* "Surface Water Supply of the United States, 1922, Pt. VIII, Western Gulf of Mexico Basins," *Water Supply Paper 548*, U. S. Geological Survey, 1925.

The construction of the original McMillan Dam—a rock-fill structure, 1 686 ft. long and 52 ft. high, with an auxiliary earth embankment, 5 200 ft. long and 19 ft. high—was completed by the Pecos Irrigation Company in the winter of 1893-94. In 1906, following the destruction of the system by floods, reconstruction of the project was undertaken by the U. S. Reclamation Service and since that time it has been successfully operated by the Federal Government. During the period from 1894 to 1903 the reservoir was operated with a spillway crest elevation of 3 258.9 ft., corresponding to a stage of 17.3 ft. on the reservoir gauge. In 1908 and 1909 the outlet works were reconstructed and the embankments repaired so that the lake level could be raised to a stage of 23.5 ft. Further enlargements were made in 1911 and 1912, and again in 1915. The present spillway crest is at Elevation 3 268.6, corresponding to a reservoir stage of 27.0 ft. Reliable topographic data of the original reservoir site were never obtained. However, such information as can be found indicates that the total storage capacity below spillway level, at the time of construction, was about 32 500 acre-ft., or about 10% of the average annual run-off at the reservoir site.

Measurements of silt accumulated in the reservoir were made by the U. S. Geological Survey in 1904, by the U. S. Reclamation Service in 1910 and 1915, and by the U. S. Bureau of Reclamation (formerly the Reclamation Service) in 1925. In the 1904 survey, which was made when the reservoir was practically empty, depths of silting were determined by auger borings, and the borings were located by plane-table.* Although some uncertainties in determining the elevation of the original ground surface must have been encountered at times, the descriptions of the survey indicate that the work was carefully done and that the final results probably possess a reasonable degree of accuracy. Later measurements of silt accumulations were made by taking cross-sections of the reservoir at intervals throughout the lengths of the silted areas, using customary surveying methods.

Table 8 contains a summary of the data on silt deposited in Lake McMillan as determined by the various surveys, total accumulations in acre-feet below certain stages being given for the periods covered by the investigations. Table 9 gives the average rates of silt accumulation, in acre-feet per year, for the different periods.

The data in Table 8 show that 41 400 acre-ft. of silt were deposited in the reservoir, below the 25-ft. stage, during the period from January 1, 1894, to June 1, 1915, a total approximately one-eighth greater than the entire original capacity of the reservoir below the 17.3-ft. stage. However, the most interesting feature shown by Tables 8 and 9 is the great reduction in silting which occurred in the reservoir proper during the period from June 1, 1915, to June 1, 1925. During this 10-year period only 3 500 acre-ft. of silt were deposited in the reservoir below the 25.0-ft. stage, giving an average rate of silt deposition of 350 acre-ft. per year as compared with an average rate of 1 930 acre-ft. per year for the period from January 1, 1894, to June 1, 1915. The reason for this marked reduction in rate of silting is that since

* Third Annual Report, U. S. Reclamation Service, 1903–04, p. 371.

about 1918 dense and extensive growths of tamarisk (also called salt cedar), have developed at the upper end of the lake. These growths have caused a spreading of the river flow above the reservoir, resulting in velocity reductions and silt deposition, and thus keeping the larger proportions of silt from entering the lake.

TABLE 8.—Summary of Data on Silt Accumulations in Lake McMillan, Carlsbad Project, New Mexico.

Time Period.		Duration, in years.	Total Silt Accumulations, in Acre-Feet, Below Reservoir Stages of:			
From.	To.		28.0.	27.0.	25.0.	17.3.
January 1, 1894.....	June 1, 1904.....	10.42	.....	.....	18 000	16 000
June 1, 1904.....	November 1, 1910.....	6.42	.....	30 000*	10 000	.....
November 1, 1910.....	June 1, 1915.....	4.58	20 000	18 500	13 400	.....
June 1, 1915.....	June 1, 1925.....	10.00	.....	.....	3 500	2 500
January 1, 1894.....	June 1, 1915.....	21.42	.....	.....	41 400	.....
January 1, 1894.....	June 1, 1925.....	31.42	.....	.....	44 900	.....

* January 1, 1894, to November 1, 1910.

TABLE 9.—Rates of Silt Accumulation in Lake McMillan, Carlsbad Project, New Mexico.

Time Period.		Duration, in years.	Average Rate of Silt Accumulation, in Acre-Feet, per Year Below Reservoir Stages of:			
From.	To.		28.0.	27.0.	25.0.	17.3.
January 1, 1894.....	June 1, 1904.....	10.42	.....	.....	1 730	1 530
June 1, 1904.....	November 1, 1910.....	6.42	.....	1 780*	1 560	.....
November 1, 1910.....	June 1, 1915.....	4.58	4 370	4 040	2 920	.....
June 1, 1915.....	June 1, 1925.....	10.00	.....	.....	350	250
January 1, 1894.....	June 1, 1915.....	.....	.....	.....	1 930	.....
January 1, 1894.....	June 1, 1925.....	31.42	.....	.....	1 430	.....

* January 1, 1894, to November 1, 1910.

The tamarisk is an evergreen shrub, or tree, originally a native of the Mediterranean regions, Western Asia, and India. Its somewhat showy pink and white flowers present an attractive appearance during the blooming period and, consequently, it was brought into this country for use as an ornamental shade tree. Tamarisk trees were propagated in the Pecos River Valley many years before they appeared at Lake McMillan. A clump of rather large trees was growing along the river east of Roswell, about sixty miles above Lake McMillan, some time prior to 1912, when the first tree was noticed at the upper end of the reservoir. The origin of the Lake McMillan growth is not known, but possibly the seed was brought down the river during flood periods, or was brought in by birds. L. E. Foster, Superintendent of the Carlsbad Project, describes the conditions at the upper end of the reservoir, under date of July 20, 1928, as follows:

FIG. 18.—TYPICAL TAMARISK GROWTH AT LAKE MCMILLAN, TEXAS.

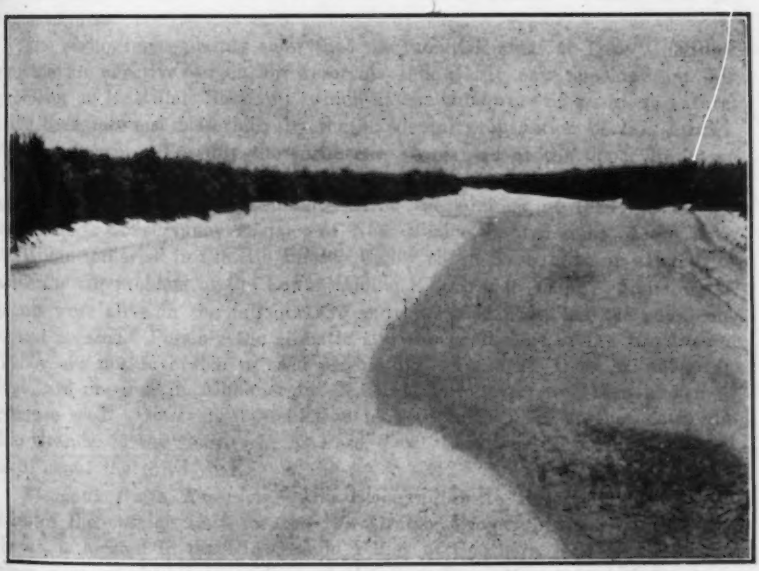

FIG. 19.—TAMARISK TREES 25 FEET HIGH ALONG THE PECOS RIVER AT THE UPPER END OF LAKE MCMILLAN.

"At the present time, the entire upper end of the reservoir is covered with a dense growth of tamarisk except for a few narrow channels. This comprises an area of about ten square miles within the flow line of the reservoir. Besides this area the tamarisk growth extends for about two miles up stream, above the reservoir flow line. This area averages about a mile wide in flood time. This flood plane is covered with the same dense growth of tamarisk as characterizes the reservoir area. The nature of the growth changes somewhat from year to year. Where water stands around the plants for more than a week, they usually die. Reforestation begins immediately after the water drains away. At the upper end of the old lake, the growth is quite uniform in age and size but as open water in the lake is approached, the growth varies considerably in size. In many places the growth is so dense as to be almost impenetrable. It ranges in height from above twenty feet down to one foot or less. The diameter of the growth ranges from the size of a pencil to six or eight inches. Just how much effect this comparatively fine, dense growth, together with the down brush, has on the velocities is not certain; but it is certain, however, that the heavier silt deposits are at the upper end of the area where such growth is most dense."

Figs. 18 and 19 are typical views of the tamarisk growth taken near the upper end of the lake on July 16, 1928. The larger trees, such as those along the river banks, are from 15 to 20 ft high. Observations at fence posts, made since 1914, show that as much as 3 ft. of silt has been deposited along the fences in some places, the greater part of which is said to have occurred during floods since 1915. Perhaps greater depths have been deposited in other places. On account of the impenetrable nature of the growth, the 1925 survey was not extended any appreciable distance into the tamarisk area.

The photographs plainly show that the tamarisk areas at Lake McMillan provide an effective screen for reservoir inflow. It now appears that silt problems at McMillan Reservoir, which at one time were of a serious nature, have been lessened materially by the accidental propagation of this foreign evergreen shrub. Possibly the shrub can be planted at the upper limits of other reservoirs, located in regions of similar climatic conditions, and there utilized to secure similar results. During the summer of 1926 the Middle Rio Grande Conservancy District of New Mexico planted some of the Lake McMillan tamarisk in the Rio Puerco Valley above Socorro, in an attempt to solve the silt problem of the Lower Middle Rio Grande Valley. Some of the plants were alive in the fall of 1927 and bore seed pods, but the pods contained no seed. Possibly the climatic and soil conditions of the Rio Puerco Valley are not favorable to seed production. Tamarisk trees, 20 to 30 ft. high, are growing in Albuquerque, N. Mex., but have never been known to produce seed. However, reproduction by seed takes place farther down the Rio Grande, in the vicinity of El Paso, Tex. Seedlings do not produce seed until about the third year.

Elephant Butte Reservoir.—The Elephant Butte Reservoir, which constitutes the storage unit for the Rio Grande Project of New Mexico and Texas, is located in the Rio Grande Valley of Southern New Mexico. The river rises on the eastern slope of the San Juan Mountains of Southern Colorado, at an elevation approximately 12 000 ft. above mean sea level;

flows east and south to the New Mexico State line; then almost due south across New Mexico to El Paso. At the Colorado-New Mexico State line the flow does not contain large proportions of sediment. However, through the State of New Mexico, tributary streams, draining large areas of arid, easily eroded soil, bring in heavy loads of material, so that by the time the Elephant Butte Reservoir is reached the flow is heavily charged with silt. The mean annual flow of the river at the upper end of the reservoir is approximately 1 200 000 acre-ft., the yearly records varying from a minimum of 240 000 to a maximum of 2 500 000 acre-ft.

The construction of the Elephant Butte Dam, a concrete gravity structure, 306 ft. high, located about 130 miles above El Paso, was completed by the U. S. Reclamation Service in 1915. Although some water was impounded in January, 1915, substantial storage did not begin until January, 1916. The reservoir above the dam is 45 miles long, has an average width of 1¾ miles, and floods a total area of approximately 40 000 acres when filled to the spillway crest, Elevation 4 407.0, on the project datum which is 43.3 ft. above mean sea level. At the time of completing the dam the reservoir had a total storage capacity of 2 638 860 acre-ft. below the spillway crest, assuming that appreciable silting did not take place within the flow line of the reservoir from the time the original surveys were made until the dam was finished. Such a capacity is approximately 220% of the mean annual run-off from the drainage area above the reservoir.

A topographic survey of the reservoir site, sufficiently accurate to determine 10-ft. contours, was made below Elevation 4 370 in 1903, and was extended to include the present flow line in 1908. Silt surveys of the upper portion of the reservoir, where practically all the silt is deposited, were made in 1916, 1920, and 1925, measurements being made by running profiles along established cross-sections spaced from ¼ to ¾ mile apart. Difficulties were encountered at times in determining the true surface of the silt deposits, due to the comparatively great depths of water, the character of the deposits, and the presence of trees, brush, or débris on the bed of the reservoir. However, the measurements were made with a great deal of care and the results are probably sufficiently accurate for the purposes intended. An exhaustive study of the chemical and physical properties of the silt was made in 1916.*

The results of the 1920 survey, as compared with the results of the 1916 survey, showed that 90 858 acre-ft. of silt were deposited in the reservoir during the intervening period of 48.7 months, corresponding to an average rate of deposition of 1 865 acre-ft. per month, or 22 400 acre-ft. per year. Comparison of the results of the 1920 survey with the original reservoir topography, taken in 1903 and 1908, indicated a total silt accumulation of 140 000 acre-ft. up to the time of the 1920 survey, practically all of which must have taken place after the reservoir began storing water. Assuming that storage began in January, 1915, the corresponding rate of silting would be 2 060 acre-ft. per month, or 24 700 acre-ft. per year. However, the Project Manager considered the values based on the original topography to be less accurate

* "Notes on Silting of Elephant Butte Reservoir," *Reclamation Record*, September, 1916, p. 423.

than those based on the 1916 survey. The total silt accumulation indicated by the 1916 and 1920 surveys amounted to 1.66% of the total inflow to the reservoir during the intervening period.

TABLE 10.—SILT DEPOSITION AT ELEPHANT BUTTE RESERVOIR.

Elevation, in feet.*	Original reservoir capacity, in acre-feet.	1925 CONDITIONS.		PERIOD, 1916 TO 1925.†		
		Total silt deposited, in acre-feet.	Remaining reservoir capacity, in acre-feet.	Total silt deposited, in acre-feet.	Rates of Silting in:	
					Acre-feet per year.	Acre-feet per month.
4 407	2 638 860	231 735	2 407 125	177 740	20 508	1 709
4 400	2 365 059	225 287	2 139 771	172 711	19 928	1 661
4 390	2 009 384	213 221	1 796 163	162 930	18 800	1 567
4 380	1 693 971	201 297	1 492 674	152 870	17 639	1 470
4 370	1 406 600	179 015	1 227 585	128 985	14 883	1 240
4 360	1 163 188	148 312	1 014 876	95 746	11 048	921
4 350	953 041	127 539	825 502	78 144	9 017	751
4 340	776 741	113 519	663 222	65 101	7 512	626
4 330	620 323	100 156	520 167	52 684	6 079	507
4 325	556 100	91 993	464 107	49 945	5 763	480

* Above project datum, add 43.3 ft. for sea-level elevations.
† Total time, 104 months.

Table 10 gives data on reservoir capacities and silt accumulations below elevations varying from 4 325 to 4 407 ft., as determined by the 1925 survey. Fig. 20 shows the effect of silting on the capacity of the reservoir.

The results of the 1925 survey indicate a total silt accumulation of 231 735 acre-ft. from the time the original reservoir topography was taken to the time of the 1925 survey, and a total accumulation of 177 740 acre-ft. during the 104 months from the time of the 1916 survey. The latter figure corresponds to an average monthly deposit of 1 709 acre-ft. and an average yearly deposit

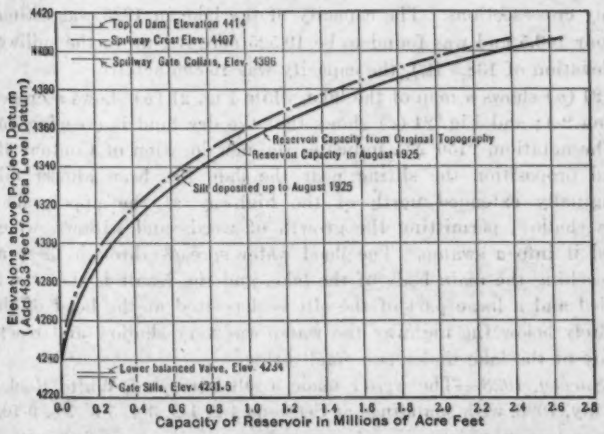

FIG. 20.—EFFECT OF SILT DEPOSITION ON CAPACITY OF ELEPHANT BUTTE RESERVOIR.

of 20 508 acre-ft. It also corresponds to 1.64% of the total reservoir inflow during the period considered, a figure almost the same as that obtained by the 1920 survey. Investigations show that 72%, or nearly three-fourths, of the silt that enters the reservoir, is deposited within an area containing not quite one-fourth of the total reservoir capacity. The total accumulation of 231 735 acre-ft. shown by the 1925 survey is only about 8.78% of the original storage capacity below the spillway crest, indicating an average yearly rate of silting of slightly less than 1% of the original storage capacity. Assuming such a silting rate to keep up indefinitely (which would be a severe assumption), the total life of the reservoir would be 114 years. Since 13 years have already passed, the remaining life would be 101 years.

William Jenner Powell,* M. Am. Soc. C. E. (by letter).—The writer has known in a general way that Lake Austin was silting rapidly, but he was not prepared for the staggering information that 95% of the water capacity had been taken up by silt in 13 years of service. The water-shed of the Colorado River for about 38 000 sq. miles spreads over what is generally known as a grassy or wooded area including, in its topographical features, flat prairie country and rugged mountainous sections.

The writer has made a survey of the silting of the White Rock Reservoir, which is about four miles east of the City Hall of Dallas, Tex. The drainage area is a wide limestone valley 114 sq. miles in extent. The reservoir was begun in 1910 and was ready for service in 1912. Practically the lowest point in the lake was 424 ft. above mean low tide. Careful surveys were not made of the original volume of water and its capacity was only estimated. The dam is of earth, and a concrete spillway with a long wide apron was constructed at the east end of Elevation 138.5, City of Dallas datum, or Elevation 457.45 above mean low tide.

Silt Surveys.—In 1923, the City Engineer made a very elaborate survey of the White Rock Reservoir, establishing bench-marks, triangulation stations, and many cross-sections. The capacity of the lake in 1923 was estimated up to Contour 140.5 and was found to be 19 525 acre-ft.; up to the spillway crest (local elevation of 138.5 ft.), the capacity was 16 896 acre-ft.

Fig. 21 (*a*) shows a map of the lake, while Fig. 21 (*b*) shows a cross-section at Section 2-4; and Fig. 21 (*c*) shows that the dry land is creeping down the lake. The notation, "136 N", indicates the new location of Contour 136. As a general proposition the silting near the dam has been almost nil. The lake originally extended north of the highway at the upper end. This part was shallow, permitting the growth of weeds and willows, which soon converted it into a swamp. The flood water spreads through these marshes before reaching the main body of the lake, and the result is that the current is retarded and a large part of the silt is deposited at the head of the lake. Immediately below the highway the water was very shallow and reached the main body of the lake under two small bridges.

Silt Survey, 1928.—The writer made a silt survey of White Rock Reservoir in July, 1928, with soundings at Sections 4-2, 1-2, 3-4, 5-6, 7-8, 9-10, 11-12,

* Cons. Engr., Dallas, Tex.

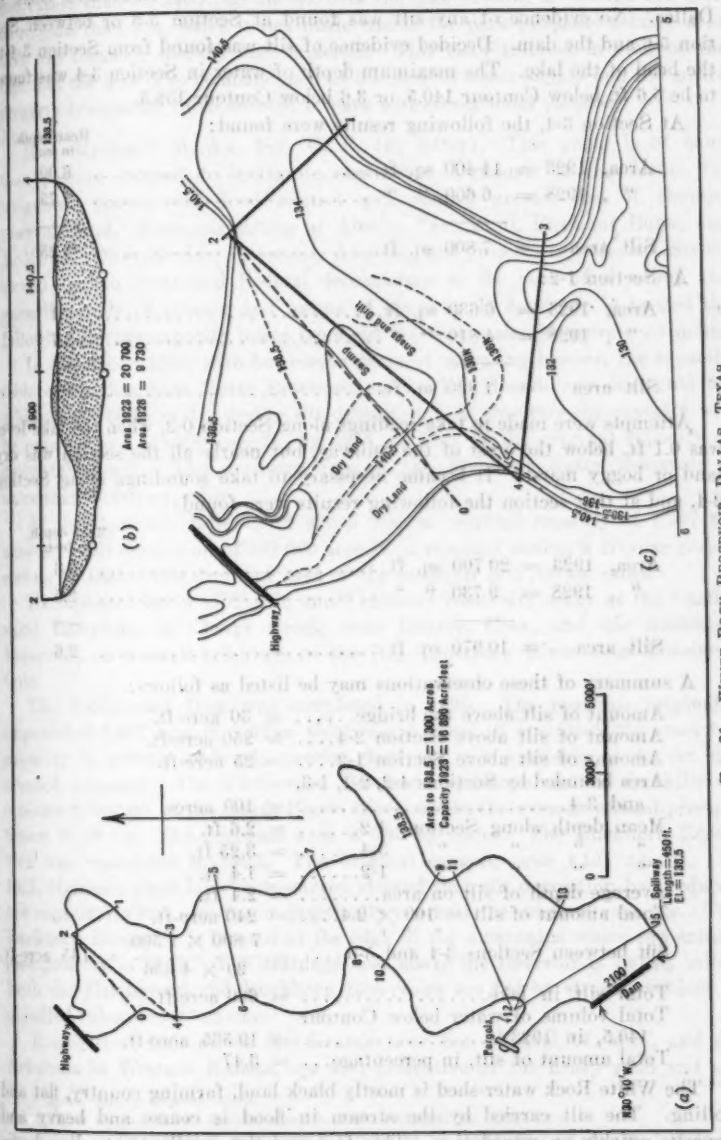

FIG. 21.—WHITE ROCK RESERVOIR, DALLAS, TEXAS.

and 12-13 (Fig. 21 (*a*)). These soundings were compared with those taken in 1924 by Messrs. McCombs and Ernest, of the City Engineer's Office of Dallas. No evidence of any silt was found at Section 5-6 or between Section 5-6 and the dam. Decided evidence of silt was found from Section 3-4 to the head of the lake. The maximum depth of water in Section 3-4 was found to be 5.6 ft. below Contour 140.5, or 3.6 below Contour 138.5.

At Section 3-4, the following results were found:

		Mean depth, in feet
Area, 1923	= 14 400 sq. ft.	6.00
" , 1928	= 6 600 " "	2.75
Silt area	= 7 800 sq. ft.	3.25

At Section 1-2:

Area, 1923	= 6 630 sq. ft.	5.1
" , 1928	= 4 810 " "	3.7
Silt area	= 1 820 sq. ft.	1.4

Attempts were made to take readings along Section 0-2, when the lake level was 0.1 ft. below the crest of the spillway, but nearly all the section was dry land or boggy marsh. It became necessary to take soundings along Section 2-4, and at that section the following results were found:

		Mean depth, in feet.
Area, 1923	= 20 700 sq. ft.	5.3
" , 1928	= 9 730 " "	2.5
Silt area	= 10 970 sq. ft.	2.6

A summary of these observations may be listed as follows:

Amount of silt above the bridge..... = 30 acre-ft.
Amount of silt above Section 2-4.... = 250 acre-ft.
Amount of silt above Section 1-2.... = 25 acre-ft.
Area bounded by Sections 4-2, 2-1, 1-3, and 3-4..... = 100 acres.
Mean depth along Sections 4-2..... = 2.6 ft.
" " " " 3-4..... = 3.25 ft.
" " " " 1-2..... = 1.4 ft.
Average depth of silt on area..... = 2.4 ft.
Total amount of silt = 100 × 2.4.... = 240 acre-ft.
Silt between Sections 3-4 and 5-6.... = $\frac{7\,800 \times 1\,500}{20 \times 4\,356}$ = 135 acre-ft.
Total silt in lake..... = 680 acre-ft.
Total volume of water below Contour 140.5, in 1923..... = 19 535 acre-ft.
Total amount of silt, in percentage... = 3.47

The White Rock water-shed is mostly black land, farming country, flat and rolling. The silt carried by the stream in flood is coarse and heavy and deposits quickly on retardation. This fact and the excellent retarding basin formed by the growth at the head of the lake have practically prevented silting

in the main body of the lake, and by making land in the shallows at the upper end rather than distributing the silt over the lake bottom, the silting has been more gain than loss, since it has made usable for park purposes a large area that was formerly marshy and valueless. This is of interest in view of the fact that the lake is to be abandoned as a source of water supply and the entire property is expected to be devoted to park and recreational purposes.

R. I. MEEKER,* M. AM. SOC. C. E. (by letter).—This paper is of more than passing interest to hydraulic engineers. Demands of the future for irrigation, power, and flood control will compel greater use of channel reservoir sites. Reservoir silting at Austin, Tex., Zuni, Elephant Butte, and McMillan,† New Mexico, Roosevelt, Arizona, and projects in adjacent States, should awaken State and Federal departments to the need of a survey and scientific study of erosion and means of control. If emphasis is needed the following storage capacity losses from silt deposition should prove convincing.

In December, 1925, with but eleven years of operating history, the capacity table of the Elephant Butte Reservoir on the Rio Grande was corrected for a loss of 220 000 acre-ft., due to silt deposition. At this rate the capacity will be reduced to 1 418 000 acre-ft., by 1975, or 54% of the original capacity of 2 638 000 acre-ft. The drainage area of the Elephant Butte Reservoir is more than 30 000 sq. miles.

At the Roosevelt Reservoir,‡ which had an original capacity of 1 367 000 acre-ft., a silt deposition of 101 000 acre-ft. is reported during a 20-year period ending in 1925. The drainage area of the reservoir is 5 760 sq. miles.

Examples of heavy silting in small channel reservoirs occur at the Castlewood Reservoir, on Cherry Creek, near Denver, Colo., and the Buckhorn Reservoir, on a small tributary of the Big Thompson River, near Loveland, Colo.

The Castlewood Dam was completed in 1890. The reservoir originally impounded 5 267 acre-ft. After thirty-eight years of operation the reservoir capacity is greatly reduced; precise data are not available as to the decreased capacity. The Castlewood Reservoir is located in the foothills on a plains tributary of the South Platte River, where the average annual precipitation is 18 in. The drainage area is 165 sq. miles. The Buckhorn Reservoir was completed in 1907. The original capacity was 1 191 acre-ft. In 1925, eighteen years later, a re-survey showed that silt deposition had reduced the capacity to 626 acre-ft., or practically one-half the original capacity. The Buckhorn Reservoir is located at the edge of the mountains where the annual precipitation is 16 in. The drainage area above the reservoir is 130 sq. miles. Both the Castlewood and Buckhorn Reservoirs are fed by streams subject to so-called "cloudburst" storms.

Rising river beds of the Rio Grande near San Marcial, N. Mex., and the Arkansas in Western Kansas, are also indicative of the heavy sand and silt

* Cons. Engr., Denver, Colo.

† Third Report, U. S. Reclamation Service, 1903-04, pp. 371-373.

‡ "Silt in the Colorado River and Its Relation to Irrigation," by Samuel Fortier, M. Am. Soc. C. E., and Harry F. Blaney, Assoc. M. Am. Soc. C. E., *Technical Bulletin* 67, U. S. Dept. of Agriculture, 1928, p. 22.

burden resulting from erosion induced by settlement. The former has risen 12 ft., and the latter 5 ft., in a period of 45 years.

Considerable information is available on quantities of silt carried by streams of the Southwest and on silt deposition in reservoirs thereon. The origin of the silt question is erosion, and the writer feels that the problem should be attacked from that angle. Information is needed on its causes and control. Over-grazing by sheep and cattle has heavily depleted the grass carpet of the plains, foot-hills, and plateau areas of the Colorado, Rio Grande, Arkansas, and Platte River Basins and has contributed greatly to erosion, which is increasing in the arid Southwest. Enormous areas of public lands are affected, and public agencies cannot ignore the problem of erosion when its far-reaching effect on future water control and utilization is realized. Erosion impinges heavily on the present and future water problems of the Western United States. Experience in arid South Africa confirms the experience in the Southwestern United States, and the following statement* appears in agreement with American history:

"In course of time burning and the concentrated grazing and tramping of sheep, goats, and cattle caused progressive desiccation. To-day, the broad valleys are drained by deep and wide river channels, and the rich grass flats are dried out and eroded. Rivers only a few feet wide a generation ago are now two or three hundred feet in width. The dense bush in the valleys has disappeared, and the river channels, now ten or twenty times the size they used to be, present an unobstructed passage to flood water entering them."

TABLE 11.—Silt Deposition, Channel Reservoirs, Southwestern United States.

Reservoir.	Stream.	State.	Drainage area, in square miles.	Original capacity, in acre-feet.	Silt Deposition.		
					Lost capacity, in acre-feet.	Period, in years.	Percentage of original capacity.
Elephant Butte	Rio Grande	New Mexico.	30 000	2 368 000	220 000	11-1915-1925	9
Roosevelt	Salt	Arizona.	5 760	1 367 000	101 000	20-1906-1925	7
Austin	Colorado	Texas.	38 000	*49 300	23 559	8-1893-1900	48
				32 029	30 552	13-1913-1926	95
McMillan	Pecos	New Mexico.	.....	28 732	12 232	10-1895-1904	42
Zuni	Zuni	New Mexico.	500	10 230	9 689	22-1905-1927	95
Castlewood	Cherry Creek	Colorado	165	5 267	†	38-1890-1927	†
Buckhorn	Buckhorn Creek	Colorado	180	1 191	565	18-1907-1925	47

* Old reservoir.

† Precise data are not available at this time.

The recent *Bulletin*† on "Silt in the Colorado River", by the U. S. Department of Agriculture, could well be a forerunner of another *Bulletin* on erosion. Through the Forest Service and associated divisions, the Department of Agriculture is well qualified to study this important problem in a comprehensive and scientific way.

* "The Principles of Irrigation Engineering," by F. E. Kanthack, pp. 103–105.

† *Technical Bulletin No.* 67, U. S. Dept. of Agriculture, 1928.

The investigational studies of erosion and silt control by the Middle Rio Grande Conservancy District in New Mexico is intelligent pioneering on a subject destined to play an important part in water conservation. The States of Arizona, Colorado, New Mexico, Texas, Utah, and Wyoming cannot afford to ignore the problem of erosion and means for its control. The Colorado River Basin and the Rio Grande Basin offer excellent opportunities for a systematic survey and study of that problem. Grazing control on the Jornado Grazing Reserve north of Las Cruces, N. Mex., may develop some valuable information concerning erosion. An interesting bibliography on silt problems, prepared by the Special Committee on Irrigation Hydraulics, has been published by the Society.* Table 11 summarizes the losses in various reservoirs from silt deposition in the Southwest.

T. U. Taylor,† M. Am. Soc. C. E. (by letter).—Practically all descriptions of the measurements made, have come from the arid and semi-arid West, and these are largely confined to the Southwest. However, the results have been valuable beyond expectation. The writer, after spending more than a third of a century in studying silting of reservoirs and in making actual measurements on existing reservoirs, has had some of his cherished ideas exploded. The first of these was the assumption on his part that there was a direct connection between run-off and the amount of silt deposited in the reservoir. The data contributed by Mr. Robinson have proved that in some reservoirs there is slight connection. A very dry, parched water-shed with a sudden run-off will produce a much greater amount of silt than a much heavier rain on a wet water-shed.

It has been claimed that lakes spreading out over their banks and having widths many times that of the river channel, will not silt as rapidly as channel lakes. This is not substantiated by experience at the old or the new lakes at Austin, Tex. The width of the new lake on the average was three times that of the river channel, two-thirds of it being over the relatively flat uplands near the dam. For the stretch extending from the dam to a point 1 003 ft. up stream the channel part of the lake has silted to 84% of its capacity in 1913, while that part on the nearly flat valley which was formerly a farm, has silted to more than 114% of the original capacity. Next to the bank on the east side there has been added (in the period from 1913 to 1926) a strip of land of an average width of 327 ft. and a length along the river bank of 1 027 ft., covering an area of 14.5 acres, all outside the channel area. On the west bank, 1 mile above the dam, a more intensive silting outside the channel area has taken place. At this point, a boathouse resting on high columns, under which there was a depth of 11 ft. of water in 1913, is now "high and dry" on 16 ft. of silt at least 200 ft. from the water's edge. In all, there are three broad marginal areas on the west side of the lake, aggregating a total length of 5.2 miles and containing silt to the amount of more than 100% of the original capacity. On the east margin there are five curved strips aggregating a total length of 7.5

* *Proceedings*, Am. Soc. C. E., March, 1925, Society Affairs, pp. 151–153.
† Dean of Eng., Univ. of Texas, Austin, Tex.

miles where the lake spreads out over old farm lands. In this total length, that part outside the channel is more than 111% in every case, while that in the channel proper is less than 88% of the original capacity.

In 1913, at a point 1 000 ft. up stream from the dam, the water was 3.2 ft. deep on the east side. In 1928, the silt was 7.8 ft. above the crest of the dam, making a total silting at this section of 11.0 ft. on the east bank. In the center of the present channel there is a depth of water of 5.5 ft. where, in 1913, there was a depth of 51.42 ft., or 45.92 ft. of silt, in the middle of the old and present channel.

A study of all the reservoirs for which data can be obtained convinces the writer that silting is no respecter of channel or valley. The broad reservoirs of McMillan, Zuni, and Lake Worth, all bear out the fact that valley reservoirs, if the width of the channel is insignificant with respect to the width of the lake, will collect silt. On the other hand, the reservoirs at Austin, Lake Pennick, and the Lower Rio Grande, bear out the fact that the channel will also collect silt.

The silt cakes and cracks on the Rio Grande near San Benito are almost exact duplicates of those at Austin. The behavior of these cakes will convince any one that the "hog wallow" prairies of Texas were produced by exceedingly dry seasons during which the ground broke open and left cracks from 2 to 3 in. wide. Later, rains would soften and break the edges which would sink into the cracks and leave a hill in the center and a hollow or depression between the cakes. While the distance between "hog wallows" is about 4 ft. from crest to crest, the silt cakes and cracks are not more than one-third of this.

The most striking example of the fact that valley reservoirs collect silt is the case of Lake Worth, near Fort Worth, Tex. The original volume of Lake Worth in 1915—when the reservoir first began functioning—was 47 177 acre-ft. In July, 1928, thirteen years afterward, the writer made a complete survey from the dam to the head of the lake, which gives the total capacity as 33 340 acre-ft. The lower sections of the lake (south of "9-Mile Bridge") were kept separate for comparison; the silting is shown in Table 12.

The main channel of the river, from its entrance to the dam, because of its meanders, is 10 miles long. Its total surface area is 112 acres, or only 3% of the total area of the lake. The total amount of silting in the 15 years has been 13 837 acre-ft., and the silting in the channel proper is proportionate to that in the valley, that is, in the lake area outside the channel. Some parts of the valley have filled 100%, while in no place has the channel been filled to the brim.

The writer has come to the deliberate conclusion that the ordinary freshets or small floods have little effect on the silting, and that it is extraordinary floods, water-spouts, tornadoes, hurricanes, and cloudbursts falling on a relatively dry water-shed that contribute the larger part of the silt to impounding reservoirs. The flood in the spring of 1922 contributed more silt to the new lake at Austin in 2 weeks than had accumulated in the previous 5 years.

However, if the silt is assumed to have been deposited at a uniform rate during the 13 years of the life of Lake Worth, the capacity in 1928 was 70.67%

of the full 1915 capacity; and, at the same rate, it will be reduced by 1941 to 49.95%, or about 23 000 acre-ft.

TABLE 12.—CAPACITY OF LAKE WORTH.

Location.	WATER CAPACITY, IN ACRE-FEET.		AREA, IN ACRES.	MEAN.	DEPTH, IN FEET.
	1915.	1928.	1915.	1915.	1928.
Below 9-Mile Bridge...	38 364	29 880	2 590	14.81	11.54
Above 9-Mile Bridge...	8 813	3 460	1 180	7.47	2.93
Entire lake.....	47 177	33 340	3 770	12.51	8.84

During November, 1928, the writer visited the Lower Rio Grande Valley with a view of seeing at first hand the conditions outlined in the discussion of Mr. Welty. In 1925, the Rio Grande, true to form, cut through one of the old sags referred to by Mr. Welty, and left the pumping plants of the San Benito Company and of Harlingen, Tex.—1¼ miles up stream—more than ½ mile from the river. The river cut across a U-shaped section, Fig. 22(a), and left the two pumping plants on the old loop of the river. This forced each company to use the old section of the river from the plant to the new river course as a forebay or feed canal for the pumps.

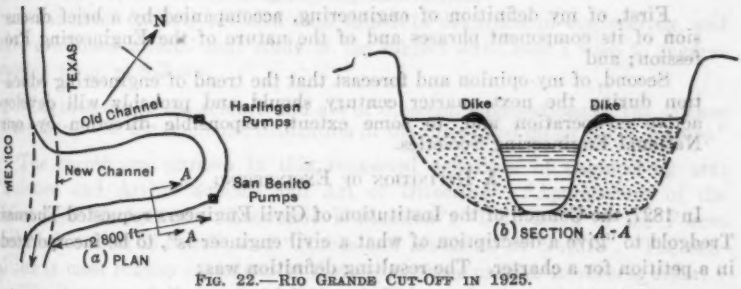

FIG. 22.—RIO GRANDE CUT-OFF IN 1925.

The silt from the Rio Grande is an ever-present menace to the San Benito Company in closing its forebay or source of supply. An 8-in. pump is kept at work in the old channel ten months of the year. The middle third of the old river is used as a channel (Fig. 22) and the "slush" is deposited on each side by the hydraulic-fill method. The pump is kept on a barge and works from it. The silt is stirred into a slush consisting of 85% water, which is pumped over a berm and is kept from flowing back into the river by a small dike on the river side of the berm. This method is used throughout the entire 2 800 ft. to the new river.

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Paper No. 1721

ENGINEERING AND ENGINEERING EDUCATION

ADDRESS AT THE ANNUAL CONVENTION, AT MILWAUKEE, WIS., JULY 10, 1929

BY ANSON MARSTON,* PRESIDENT, AM. SOC. C. E.

This address will be devoted to the presentation:

First, of my definition of engineering, accompanied by a brief discussion of its component phrases and of the nature of the Engineering Profession; and

Second, of my opinion and forecast that the trend of engineering education during the next quarter century should and probably will develop active co-operation and, to some extent, responsible direction by our National Engineering Societies.

A DEFINITION OF ENGINEERING

In 1827, the Council of the Institution of Civil Engineers requested Thomas Tredgold to "give a description of what a civil engineer is", to be incorporated in a petition for a charter. The resulting definition was:

"Civil Engineering is the art of directing the great sources of power in Nature for the use and convenience of man, as the means of production and of traffic in states both for external and internal trade, as applied in the construction of roads, bridges, aqueducts, canals, river navigation and docks, for internal intercourse and exchange, * * *, and in the construction and adaptation of machinery, and in the drainage of cities and towns."

The many attempts made by engineers during the last forty years to formulate a new definition of engineering prove that this famous statement incorporated in 1827 in the charter of the (British) Institution of Civil Engineers is not satisfactory under present-day conditions.

The difficulties in the way of devising a satisfactory definition of modern engineering are very great, perhaps insuperable. Many objections, at least

* Dean of Eng., Iowa State Coll., Ames, Iowa.

some of which appear valid, have been raised to every suggested definition of which I know.

In 1927, the Mining and Metallurgical Society of America, presented an original and an amended definition of engineering,* both prepared by a committee, together with the opinions thereon of 95 members. The definitions were as follows:

Original Definition.—

"Engineering is the practical application of scientific methods to the utilization of the resources of Nature for the use of Man."

Amended Definition.—

"Engineering is the application of scientific methods to the utilization of the resources of Nature."

Only 7 members approved the original definition and only 34 the amended definition, while 44 members were not satisfied with either. Many thought the definitions basically wrong, because they would include many occupations other than engineering.

In the years 1906-1919, inclusive, I gave annually to engineering seniors a course of lectures, about half of which were printed and copyrighted in 1912, on the History of Engineering, containing some discussions on definitions of engineering. In 1913-1914, I served as Chairman of a committee which made a study of original source material and formulated definitions of mechanic arts and of engineering, which were adopted officially and published in 1914 by the Land Grant College Engineering Association.

I was never entirely satisfied with this 1914 definition of engineering, and have continued intermittent study of the subject until now I wish to suggest another form, as follows:

"Engineering is the science and art of directing applications of the science of mechanics in the economic utilization of the forces and materials of Nature."

The significant phrases in this suggested definition of engineering are: "Science and Art"; "Science and Art of Directing"; "Applications of the Science of Mechanics"; "Economic Utilization"; "Utilization of the Forces and Materials of Nature"; and "Forces and Materials of Nature". The definition is most readily explained by discussing these phrases in reverse order.

The Forces and Materials of Nature.—This phrase covers the entire physical universe and, hence, it includes the entire field of the physical sciences and applied mathematics. While engineering is founded mainly on the sciences of mathematics, physics, and chemistry, in many instances it is based on biological science.

Engineering, however, does not include all applications of the physical and mathematical sciences, but is restricted to the particular ones that are characterized by the necessary use of designs, methods, and devices which are based on the science of mechanics.

It is a common fault of suggested definitions of engineering to word them so broadly that while often they may describe engineering with much truth, they include various other occupations which are widely different.

* *Bulletin 190*, Min. and Metallurgical Soc. of America, October, 1927.

The Utilization of the Forces and Materials of Nature.—This phrase covers not only engineering, but agriculture, the handicrafts, medicine; in short, all industries, all trades, and all professions, in which any use is made of any of the organic or inorganic forces or materials of the physical universe.

Engineering, however, does not include all utilizations of the forces and materials (the resources) of Nature, but is restricted to those particular utilizations which are characterized by applications of the science of mechanics.

Economic Utilization.—Emphasis has always been given the economic side of the engineer's work, his ability "to do with one dollar what some one else could do with two". In recent years this has been made more emphatic, and it is now widely recognized that the engineer must have a working mastery of the correct principles of economics and business. Engineering is now recognized as including the work of the industrial executive.

Applications of the Science of Mechanics.—This phrase covers all mechanic arts industries. In 1914, the Land Grant College Engineering Association officially adopted the following definition:

"The mechanic arts are those arts which are characterized by applications of the science of mechanics."

It was prepared by the Executive Committee of the Association, after a careful study of original source material, some of which, arranged in chronological order, is quoted as follows:

"'The use of the term, Mechanic Arts, in standard literature dates back to Francis Bacon, 1561-1626. The term was first applied to machine devices.'" (Letter from the Lexicographer, *Literary Digest*, 1913.)

"'The Mechanick Art is a Science which contemplates about the quantities of moving forces, and the times in which the motion is made.'" ("Mechanick-Powers", Venturus Mandey and James Moxon, London, 1696.)

"'Most of what are usually called the Mechanic Arts are partly mechanical and partly chemical.'" (Webster's Dictionary, 1848; copyright date, 1847; on the title-page, 1848.)

"'Journal of the Franklin Institute of the State of Pennsylvania, for the Promotion of the Mechanic Arts, Devoted to Mechanical and Physical Science, Civil Engineering, the Arts and Manufactures, and the Recording of American and Other Patent Inventions.'" (Title of the *Journal* for 1857, Philadelphia.)

"'Mechanics, Applicate or Applied, is a term which, strictly speaking, includes all applications of the principles of abstract mechanics to human art, * * *.'" (Encyclopedia Britannica, 1857, American Reprint, Little, Brown & Co., Boston.)

"'The Mechanic Arts are those which comprehend the means of promoting and facilitating the necessities of existence.'" (The American Encyclopedia, 1859-63.)

"'Mechanical Art is most easily distinguished from fine art by the character of its products: Those of fine art represent ideas, those of mechanical art answer purposes; the former result in a gratification of the æsthetical sense, the latter are made for practical use. * * *'". (Johnson's New Universal Cyclopedia, 1877, Edited by F. A. P. Barnard, President of Columbia, and Arnold Guyot, Professor of Geology and Physical Geography, College of New Jersey.)

No permanent list of mechanic arts industries can be made, for new industries are constantly developing as civilization advances. A present-day list would include:

Ceramic Industries.
Communication Industries: Telegraph, Telephone, Wireless.
Construction Industries.
Electrical Industries.
Gas Industries.
Handicrafts: Masonry, Metal, Wood, etc.
Manufacturing Industries.
Mining Industries.
Petroleum Industries.
Power Industries: Electric, Gas, Steam, Water, Wind.
Quarry Industries.
Sanitation Industries: Sewerage, Water Supply.
Transportation Industries: Aeronautics, Highways, Navigation, Railways, Waterways.
Utilities: Bus, Electric, Gas, Heat, Railway, Water.
Various other industries not enumerated.

Agriculture is not a mechanic arts industry, although it requires the use of many mechanical devices, for agriculture is characterized by the cultivation and care of plants and animals, not by the use of tools and machines. Many other industries and occupations in which the use of mechanical contrivances is essential, are excluded for similar reasons.

Only a limited fraction of the hundreds of millions of persons who are employed in the mechanic arts industries can properly be termed engineers.

The Science and Art of Directing.—This phrase restricts engineering to the professional phases of mechanic arts work.

Tredgold was right in making the phrase, "the art of directing", one of the most prominent components of his famous definition, and the word, "directing", should be considered an essential part of any correct definition of engineering.

Engineering is a true profession, with real professional characteristics and requirements, including:

1.—A great system of engineering education.
2.—Increasingly widespread requirement of governmental professional licenses to practice engineering.
3.—A great professional engineering literature.
4.—Many local and several great National professional engineering societies in each great country.
5.—Increasingly strict enforcement of formal codes of professional engineering ethics.

Engineering differs in characteristics from the other great professions mainly in that the engineer is unique in his close and vital relation to hundreds of millions of persons engaged in pursuits closely allied to his profession. It is easy to make the distinction between the apothecary and the physician, or between the bailiff and the lawyer, but it is not so easy to draw a sharp line of separation between the highly skilled mechanic arts master workman and the professional engineer. This is because engineering is both a science and an art.

Science and Art.—Science is classified knowledge; "art is that which depends on practice and skill in performance". "In science, we know that we may know; in art, we know that we may produce."*

* See Webster's Dictionary, and Karslake as quoted therein.

From the remotest ages, engineering has originated from the union of the most advanced contemporaneous physical and applied mathematical science with the highest contemporaneous skill attained in practice in the mechanic arts. As far back as the beginning of the Christian era, the Roman architect-engineer, Vitruvius,* wrote of architecture, which then included engineering:

"Architecture is a science arising out of many other sciences, and adorned with much and varied learning; by the help of which a judgment is formed of those works which are the results of other arts. Practice and theory are its parents. * * * the mere practical architect is unable to assign sufficient reasons for the forms he adopts; and the theoretic architect also fails, grasping the shadow instead of the substance. He who is theoretic as well as practical, is therefore doubly armed; able not only to prove the propriety of his design but equally to carry it into execution."

In Tredgold's day (1827), almost all English engineers had risen from the ranks of master workmen, and it was natural for him to refer to engineering as only an "art". Since Tredgold's time, there has been a very extensive development of the scientific aspects of engineering knowledge and qualifications, which I have come to believe should be recognized by defining engineering as both a "science" and an "art". While engineering is based on certain phases of the physical and mathematical sciences, there is a body of real "engineering science," consisting of certain highly specialized developments (dealing almost wholly with engineering problems) of the physical and mathematical sciences. Engineering mechanics, the theory of structures, and the theory of machine design are good illustrations. The physicist, the chemist, the biologist, the mathematician, would have to become engineers to become qualified to develop and advance modern engineering science.

More and more universally in modern engineering, empirical methods and solutions of engineering problems are found insufficient and unsatisfactory. No engineering problem can be considered solved satisfactorily until a sound theory of its solution is discovered.

The origin of the words, "engineering" and "engineer", is such as to indicate that the engineer should be thought of as primarily a man of genius, or possessing ingenuity, and that engineering should not be considered as merely an art.

The words† "engineering" and "engineer" come from the Sanskrit, *jan*, to be born, from which came the Greek form, *γεν*, and the Latin, *gen*.

"The direct line of derivation [of the word, engineer] may be traced to *ingenium*—natural capacity, talent, capacity for invention—and is closely linked with the words *genius* and *ingenuity*."‡

The name, "engineer", came to us from the Latin through the French noun, *ingénieur*.

It is a mistake to think that the name of our profession is derived from the word, engine, or that the engineer is primarily concerned with engines. An engineer is an ingenious man, a man of genius. An engine is an ingenious machine.

* Translation by Joseph Gwilt, an English architect, 1874.

† "Life of Sir William Fairbairn," by Dr. William Pole (Abridged), *Minutes of Proceedings*, Inst. C. E., Vol. CLXXIX, 1909-10.

‡ *Bulletin No. 16*, Soc. for the Promotion of Eng. Education, 1929.

Engineering Education—Its Relation to Professional Engineering Organizations

No profession can be a true profession and keep up with the advance of science without wise, ample, and ever-improving provision for its professional educational requirements. In the United States, our engineering societies have not taken a sufficiently active part in shaping and directing engineering education. In a paper* entitled "A Comparative Study of Engineering Education in the United States and in Europe", Mr. W. E. Wickenden gives an excellent summary discussion of the development of modern engineering education in Europe and in the United States. The time has come when our engineers' clubs within the great cities, our State engineering societies within the several States, and our great National engineering societies for the Nation, ought to assume real responsibility and undertake real initiative in engineering education.

It is not sufficient for engineering practitioners to confine themselves to criticism of engineering graduates; that is too much like school boys making faces at their professors from behind their backs.

The writer agrees with Mr. Wickenden* that:

"In no other country [than the United States] have the engineering schools been so free from outside domination. They owe little to statecraft other than the provision of means for their extension and support. They owe little to the organized engineering profession except the benefits of occasional criticism of their aims and methods. They owe little to industries except an ever-widening field of employment for their graduates. To a striking degree, these schools have been left to work out their own destiny and fend for themselves. This has been the source of both weakness and strength."

It should be borne in mind that throughout all civilizations, up to the beginnings in the Eighteenth Century, in France, of the development of our modern engineering schools, the education of engineers was left almost wholly to themselves. We know by the character of their engineering constructions that the great engineers of Ancient Chaldea and Egypt must have been fully acquainted with the best scientific and mechanics arts knowledge of their times. We know that Heron, of Alexandria, whom Truesdell† has erroneously designated "the first engineer", was a great mathematician and author, as well as a great engineer. The Roman architect-engineer Vitruvius, enumerates the subjects in which an architect should be educated, as including: Good writing (we now say "English"); history; music; moral philosophy (ethics); physics (sanitation); law (business law relating to party walls, roof water, cesspools, sewers, and the like); natural philosophy (physics, as exemplified in the works of Archimedes and Ctesibus and as applied to the conduct of water); drafting; arithmetic; geometry; and astronomy.

In modern times, engineering education in England was left mainly to the Engineering Profession itself for a long time and, according to Mr. Wickenden,*

* "A Comparative Study of Engineering Education in the United States and in Europe", *Bulletin No. 16*, Soc. for the Promotion of Eng. Education, 1929.

† *Journal*, Assoc. of Eng. Societies, Vol. 19, July, 1897.

"From its foundation in 1818, the Institution of Civil Engineers has exercised a dominant influence on the professional training of engineers. British policy is to leave such matters to the organized profession rather than the law and the professional institution, by virtue of its character, acts as a quasi-legal body."

In Germany, says Mr. Wickenden,*

"The influence of the *Verein Deutscher Ingenieure* must be ranked among the major forces in the upbuilding of both the technical universities and the *realschulen*."

It is true that America is not England or Germany. The reason for our comparative failure to the present time to develop adequate systems of elementary trade schools and technical high schools is the universal freedom to choose any occupation and to aspire to any social plane which our American ideals strive to preserve for the American youth until they have passed the high school age. Our engineering schools are not to blame, as some have charged. Those engineering schools which have failed to devote their energies mainly to professional college courses have been killed or dwarfed by lack of attendance. You cannot teach without students.

The fact that the problem of developing adequate systems of technical education is vastly more difficult in America than in countries where class distinctions lead the great majority of youth to settle inevitably at comparatively early ages into defined life occupations, is not a valid excuse for American professional engineering societies to neglect their duty to assume proper responsibility and initiative in connection with engineering education. By co-operation with the Society for the Promotion of Engineering Education the American Society of Civil Engineers should proceed promptly to assume real responsibilities in shaping civil engineering education in our engineering colleges. In this, as in other engineering activities, the Society should work in co-operation with the other Founder Societies, as far as they are ready to go.

In my judgment, the Society might well at this time study with great care the feasibility and advisability of proceeding in the immediate future with active co-operation in shaping and advancing civil engineering education in America in accordance with all or a part of the following program, several features of which have already been given tentative consideration by the Board of Direction:

1.—Adopt formal requirements, materially higher than have been accepted heretofore, for the admission of engineering colleges to the accredited list of the Society. In formulating the standards, the Society should consider contents of engineering curricula, qualifications of engineering professors (which will necessarily involve their remunerations), engineering libraries and their use, housing, laboratory equipment, engineering research, and the achievements of engineering alumni.

2.—Require candidates for admission who are non-graduates of accredited engineering colleges, as a condition for acceptance, to pass actual examinations demonstrating educational and professional attainments equivalent to the completion of a standard engineering course. In such examinations (which should

* "A Comparative Study of Engineering Education in the United States and in Europe", *Bulletin No. 16*, Soc. for the Promotion of Eng. Education, 1929.

be partly written and partly oral), the candidate's professional achievements should be given due weight. Questions for the written examinations should be prepared and the papers should be graded at Society Headquarters. The examinations might be given by the Local Membership Committees.

Candidates who are graduate engineers might well be required to appear in person before the Local Membership Committees for conferences which would be somewhat of the nature of oral examinations.

3.—In my judgment, the Society might very properly and wisely proceed, in co-operation with the Society for the Promotion of Engineering Education, to formulate and secure the adoption, by accredited engineering colleges, of standard requirements for the professional engineering degree of Civil Engineer (C. E.).

Almost all our standard engineering colleges are already agreed that the professional degrees, of Civil Engineer, Mechanical Engineer (M. E.) and the like, should be awarded only after years of successful engineering practice; never for engineering college work alone, for which the standard degree is Bachelor of Science (B. S.). It is, therefore, very proper that our National Civil Engineering Society participate in deciding how much and what kind of civil engineering practice should be required before giving any person the legal right to use the title of Civil Engineer.

The present requirements for the degree of Civil Engineer vary greatly in different institutions, and I believe that the engineering colleges would, in general, welcome action by the Society which would standardize professional degrees. When so standardized, the Society might well give such degrees greater weight than at present in connection with its requirements for admission to the grade of Associate Member.

4.—The Society ought to take some active steps to aid effectively in the development of true graduate civil engineering work of high order in our engineering colleges.

It now seems settled that, at least for a considerable period in the future, the standard engineering college course will be four years, following graduation from standard secondary schools. This means that the educational attainments of the engineering graduate cannot be expected to advance markedly until that indefinite future date when the junior high school and the junior college movements develop to the point of delivering to the engineering colleges candidates for admission who have already completed one or two years of work now done in college.

The engineering colleges, however, are now actively developing one to three years of graduate engineering work, designed to serve only the best fraction of engineering graduates. The National Engineering Societies can and should do much to assist in placing such graduate work on a very high plane, and in encouraging practicing engineers to take advantage of the facilities of the graduate laboratories of our great engineering colleges.

5.—It would be well for the Society to study the feasibility and the advisability of sponsoring in the near future advanced schools, of several weeks'

duration, restricted to practitioners of experience and merit, for instruction in the most advanced developments in specialized branches of civil engineering practice.

I visualize such schools as financed by fees and endowments and as gatherings where our most aspiring and promising civil engineers of ages, say, between 27 and 40, could learn at the feet of the foremost engineers, with some of the foremost scientists interpreting the latest scientific discoveries.

I dare even to dream that the Society might obtain from Congress special legal authority to conduct such schools, to award diplomas for completion of work therein, and possibly to confer upon that fraction of the membership of the Society who meet certain specified high qualifications some legal designation which will win them, in fact if not in law, some special rights and privileges in their professional practice.

MEMOIRS OF DECEASED MEMBERS

FREDERIC VAUGHAN ABBOT, M. Am. Soc. C. E.*

DIED SEPTEMBER 26, 1928.

Frederic Vaughan Abbot was born in Cambridge, Mass., on March 4, 1858. He was a son of Henry L. and Mary Susan (Everett) Abbot, his father having been a Brigadier-General in the United States Army, and an officer of distinction both as a Civil and as a Military Engineer.

Mr. Abbot was educated at private schools until he received an appointment to the United States Military Academy at West Point, N. Y., to which institution he was admitted as a Cadet on July 1, 1875. He was graduated in 1879, at the head of his class, and was commissioned a Second Lieutenant, Corps of Engineers, U. S. Army.

He was promoted to the grade of First Lieutenant on June 17, 1881; Captain, on July 22, 1888; Major, on July 5, 1898; Lieutenant-Colonel, on September 9, 1906; Colonel, on June 24, 1909; and Brigadier-General, National Army, on August 5, 1917. He was placed on the retired list on account of physical disability incurred in the line of duty on May 10, 1920, after more than forty years of commissioned service, all of which was with the Corps of Engineers, U. S. Army.

General Abbot had a varied and extensive career of service both as a Civil and as a Military Engineer. After graduating from West Point he was assigned to duty at the United States Engineer School of Application, Fort Totten, New York Harbor. While attending this school he translated and published for the instruction of student officers, Colonel Schiednagel's book on "Submarine Mines" (Spanish), and Schlicting's "Navigable Non-Tidal Rivers" (German). From June, 1882, to August, 1884, he served as Assistant on the improvement of the Mississippi River from St. Louis, Mo., to Cairo, Ill., and on detached service, surveying a portion of the boundary line between Maryland and Virginia. In August, 1884, he became Assistant on the improvement of rivers and harbors in North Carolina and South Carolina, and on the construction of fortifications along the South Atlantic Coast. From May, 1888, to September, 1897, he was in charge of these works, completing the project for obtaining a channel of 21 ft. across the bar at the entrance to Charleston, S. C., Harbor, by means of converging jetties; constructing a battery for sixteen 12-in. mortars on a swamp foundation; and constructing emplacements for 10-in. guns operating on disappearing gun carriages.

From September, 1897, to August, 1900, General Abbot (then a Major), was in charge of the river and harbor works of Minnesota and Wisconsin, constructing two large reservoir dams at the head-waters of the Mississippi River, and a lock and dam in that river above St. Paul, Minn.

From August, 1900, to April, 1910, he served as Assistant to the Chief of Engineers, U. S. Army, with station at Washington, D. C. In April, 1910,

* Memoir prepared by C. McD. Townsend, M. Am. Soc. C. E.

he was placed in charge of the Boston, Mass., Engineer District, and from June, 1913, to October, 1917, he served as Division Engineer of the Northeast Division Engineer Department, and was in charge of river and harbor construction in portions of New York and New Jersey.

From October, 1917, until his retirement, he again served as Assistant to the Chief of Engineers, acting as Chief of the Division of Operations, Office of the Chief of Engineers, where he supervised the enlistment, organization, training, and forwarding of more than 300 000 engineer troops for service in the World War. He also acted as Chief of Engineers on numerous occasions. During this service General Abbot was a member of numerous boards convened by the War Department or the Chief of Engineers, of which the most important were the Board of Engineers for Fortifications, the Board of Engineers for Rivers and Harbors, the New York Harbor Line Board, the Boston Harbor Line Board, the Joint Artillery and Engineer Board to locate fire-control stations at all seacoast defenses in the United States, and a Board appointed to co-ordinate the work of the Corps of Engineers, Ordnance Department, and Signal Corps.

While General Abbot's services in the field as a Civil Engineer were highly creditable, he obtained his greatest reputation as a Military Engineer and as an Administrative Officer, having been awarded a Distinguished Service Medal by the War Department for his service during the World War, the citation reading as follows:

"For exceptionally meritorious and distinguished service in the organization of Engineer troops and the procurement of enlisted men for the service in the War. His zeal was untiring and the success of his efforts marked."

The closing sentence of this citation can be applied to his entire career, "His zeal was untiring and the success of his efforts marked."

He died on September 26, 1928, and is survived by his widow, *née* S. Julia Dehon, two daughters, Marion B. and Elinor R., and a son, Henry Dehon Abbot.

General Abbot was elected a Member of the American Society of Civil Engineers on December 3, 1884.

HENRY SEWALL ADAMS, M. Am. Soc. C. E.*

DIED AUGUST 21, 1927.

Henry Sewall Adams, the son of Charles Sylvester and Christiana Wyman (Jennison) Adams, was born in Cambridge, Mass., on March 16, 1864. At an early age he lost his parents, as well as his only brother and sister, and his boyhood was spent with his Grandfather Adams, in Cambridge, and at the farm of his Grandfather Jennison, in Burlington, Mass.

At the age of seventeen, Mr. Adams began his engineering career with a survey party on the sewerage system of Woburn, Mass., work which occupied four summer vacations during his course at the Woburn High School and following his college Freshman year. In 1883 he entered the Massachusetts Institute of Technology where he studied civil engineering with the Class of 1887, leaving, however, before graduation, to take a position in the office of the City Engineer of Cambridge.

Municipal engineering work in Cambridge and work as Assistant County Engineer of Middlesex County occupied Mr. Adams until 1893, when he opened his own office in Boston, Mass., for the private practice of engineering which was continued uninterruptedly until his death.

In his early years of consulting practice, Mr. Adams' professional work included water-front developments at Boston, reporting on mines in Montana, Idaho, California, and Mexico, and on other properties in different parts of the country. In 1895 was begun the construction of Boston's Subway System, the pioneer undertaking of its kind in the United States. This extensive work of underground construction presented new and unusual problems relative to the underpinning and safety of buildings. In the several years during which subways and tunnels were being built in down-town Boston, Mr. Adams, more often than any other engineer, was called upon by private clients to advise on these foundation problems and to look after the safety of more than one hundred important buildings.

Gradually, work on difficult foundations, and more particularly that in connection with the building of water-front structures, came to absorb Mr. Adams' attention. In this special field he made a reputation which was not confined to Boston. Not only did he come to be looked upon as the leading consulting engineer on the development of the Boston water-front—and no engineer has been more closely identified with private developments at that port—but his services were in demand elsewhere. He was one of two engineers commissioned by the City of Newark, N. J., in 1909, to study and report upon the layout of the port development of the Newark Meadows; he designed wharves and docks for Ponce, Porto Rico; he designed and supervised the building of the Sandy Bay improvements at Rockport, Mass.; and he was Engineer on harbor and water-front improvements at other ports.

For many years, until his death, Mr. Adams was Consulting Engineer to the Atlantic Sugar Refineries, Limited, of Montreal, Que., Canada., on its water-front plant at St. John, N. B., Canada, where his advice was sought

* Memoir prepared by Frederic H. Fay, M. Am. Soc. C. E.

not only on difficult water-front problems, but also on the general development and upkeep of the plant. Likewise, for many years he was Consulting Engineer to the Pocahontas Fuel Company, Incorporated, in connection with its coal-handling plants at Portland, Me., and at New Bedford, Mass. At both Portland and St. John, he successfully met some unusually difficult problems of water-front construction in connection with structures, previously built by others, which had failed. Many fuel companies engaged his services in connection with the building and repair of coal-handling plants. Included among his clients were railroad and steamship companies and many corporations, as well as most of the private wharf owners along the Boston water-front.

Mr. Adams showed peculiar ingenuity and resourcefulness in solving problems of unusual difficulty and complexity. Painstaking and thorough in his work, he was also a man of broad outlook and sound judgment. By his clients, many of whom he served continuously over a period of years, his advice was sought, not only on structural problems, but also on the conduct of business.

For more than thirty years, and until his death, Mr. Adams lived in Arlington, a suburb of Boston. He retained also the farm of his Grandfather Jennison in Burlington, where part of his boyhood had been spent and in which he always took a special interest. He served the Town of Arlington in many ways. During the early years of his private practice, and until the appointment of a full-time Town Engineer, he acted in that capacity and took an important part in the laying out and development of this growing community. He was elected a member of the Board of Public Works of Arlington in 1912, a position which he filled for several years, part of the time being Chairman of the Board. He was elected a member of the first Planning Board of the town in 1914.

Being deeply interested in historical and genealogical matters, he was long an active member of the Arlington Historical Society as well as of the Bostonian Society.

Mr. Adams was admitted to membership in the Boston Society of Civil Engineers on October 19, 1892, and served as a Director of that Society in 1925 and 1926. He was a life member of the International Association of Navigation Congresses; a member of the American Society for Testing Materials, and of the Association of Professional Engineers of the Province of New Brunswick, Canada. He was a member of the Engineers Club of Boston and of the Boston Chamber of Commerce, in which latter organization he took an active part in the work of the Maritime Association.

In October, 1892, Mr. Adams was married to Bessie Lockwood Tucker, of Cambridge. He is survived by his widow and four sons, Charles Jennison, Laurence Wilder, Ralph, and Henry Sewall, Jr. There are also two grandchildren, Jane and Charles Jennison, Jr.

By ability and achievement, he worthily earned a position of standing in the Engineering Profession. Ever modest, genial, and true to his own high standards, he endeared himself to a wide circle of friends and associates. Especially was he held in high regard by the clients whom he served. One of them writes:

"Mr. Adams had the happy faculty of always impressing his client with explicit confidence in his judgment, and I never knew a more careful and painstaking man than he when he had a difficult problem confronting him, and he had one of the most lovable natures that I ever knew in a man of his ability. Our Company has suffered a great loss in the death of Mr. Adams."

Another client says of him,

"The recital of Henry S. Adams' services to this company would be a long and unbroken record of valuable contributions. He possessed an engineering ability that in itself was sufficient to distinguish him, but over and above this was a pride and joy in his work and a simple devotion to an ideal which, combined with his professional ability, gave him that place and position which he occupied so well. Possessed of a delightful modesty and never superior to gaining information from whatever source it might be offered, he none the less inspired full reliance by his own quiet confidence.

"It is, however, impossible to consider Henry S. Adams except it include his own charming and lovable character. The start of our business relationship marked also the beginnings of a personal attachment which grew and deepened until it became one of life's delightful realities. It is a great privilege to have had him as a friend and to continue him in our affectionate remembrance."

An able engineer and useful citizen, Henry Adams was a lover of Nature, a devoted husband and father, and a loyal and understanding friend.

Mr. Adams was elected a Member of the American Society of Civil Engineers on June 6, 1906.

ROBERT PAUL ANDERSON, M. Am. Soc. C. E.*

DIED JANUARY 24, 1929.

Robert Paul Anderson, the eldest son of the Rev. R. E. and Alice B. Anderson, was born in Redwood Falls, Minn., on December 11, 1884.

Mr. Anderson received his preparatory education in the grade schools of Heyworth, Ill., the Chambersburg Academy, Chambersburg, Pa., and Grand Prairie Seminary, Onarga, Ill. In the fall of 1906 he entered the University of Illinois, Urbana, Ill., where he spent three years studying Civil Engineering.

After leaving the University Mr. Anderson entered the employ of the Illinois Central Railroad Company and was engaged in the construction of docks in New Orleans, La., and railroads in Mississippi. Later, he spent a year with the United States River and Harbor Commission on Mississippi River work, and from there he went to the Spokane, Portland, and Seattle Railway Company as Resident Engineer in charge of heavy construction along the Snake River.

In 1909, Mr. Anderson was appointed Locating Engineer for the Northern Pacific Railway Company, working in Montana and Washington. He then became Assistant Engineer on Construction, the youngest man to occupy this position in the entire System.

In 1912, he became Locating and Reconnaissance Engineer for the Farquhar Syndicate in South America and three times crossed the Andes in charge of survey parties on important and dangerous work. He had many adventures in little known parts of Brazil, Bolivia, and Chile.

On returning to the United States in 1914, Mr. Anderson was engaged in construction work in El Paso, Tex., and Juarez, Mexico, until the outbreak of the World War, when he enlisted and served in France as Captain of Company E, 55th Engineers, until he was mustered out in July, 1919. During his service in France he was critically ill with pneumonia. This undoubtedly weakened his constitution and contributed to his early death.

Following the war he returned to El Paso and was engaged in work for the City Engineering Department, a notable accomplishment being the construction of the Scenic Drive on Mount Franklin. In 1923, he was appointed City Engineer and served for four years, during which time he had charge of the design and construction of a high-line reservoir of 2 300 000 gal. capacity, with a reinforced concrete dome 140 ft. in diameter, reported to be the largest flat dome in the country. Important street improvements, river protection, and sewerage projects were also carried out during this term of service.

In 1927, Mr. Anderson returned to the contracting business and was Superintendent of Construction for the Ware Company of El Paso until the beginning of his last illness.

* Memoir prepared by W. E. Stockwell, Esq., El Paso, Tex.

Mr. Anderson had a winning personality and is mourned by a host of friends from among those who came in contact with him.

In 1919, he was married to Mary Emma Jackson, of Paragould, Ark. He is survived by his widow and one daughter, Mary.

Mr. Anderson was elected a Member of the American Society of Civil Engineers on January 18, 1926.

WALTER EUGENE ANGIER, M. Am. Soc. C. E.*

DIED DECEMBER 29, 1928.

Walter Eugene Angier was born at Fitzwilliam, N. H., on May 18, 1863. His father, Philip Doddridge Angier, a native of Fitzwilliam, represented his district in the State Legislature for two terms. His mother, Sarah Arabella (Reed) Angier, a sister of the mother of the poet, Eugene Field, was the "Aunt Belle" to whom Mr. Field dedicated his "Love Songs of Childhood", and it was at his request that her son was given his name.

The family moved to West Swanzey, N. H., when Mr. Angier was very young, and it was in that town that he was reared, receiving his early education in the common schools and at Mt. Caesar Academy. In 1885, he received his Bachelor of Science degree from the New Hampshire College of Agriculture and Mechanic Arts; and, in 1887, he received the degree of Civil Engineer at the Thayer School of Civil Engineering, Dartmouth College, Dartmouth, N. H.

His first professional experience, during the summer vacation of 1886, was in the office of the late John W. Ellis, M. Am. Soc. C. E., Chief Engineer of the Providence and Worcester Railroad Company, and also with Mr. Dana C. Barber, of Philadelphia, Pa., whom he assisted in surveys and plans for the sewerage system of Lakewood, N. J.

Immediately after his graduation, in April, 1887, Mr. Angier went to Fort Madison, Iowa, as Rodman and Assistant on the Mississippi River Bridge which was being built by the Atchison, Topeka and Santa Fé Railroad Company, under the late Octave Chanute, Past-President, Am. Soc. C. E., as Chief Engineer. In this position, he inspected cement, assisted in the field work, and in the sinking and sealing of, the caissons. From October, 1887, to October, 1888, and again in January and February, 1889, he was Sub-Inspector on the harbor improvement at Galveston, Tex., and Recorder on a survey of the harbor and Lower Bay at Galveston. In the interim, from October, 1888, to January, 1889, as United States Assistant Engineer, he was engaged in an examination of the Rio Grande River, from Embudo, N. Mex., to El Paso, Tex., in order to determine its facilities for navigation.

Bridge work was Mr. Angier's particular desire and he left the Federal Service in February, 1889, to go to Memphis, Tenn., as Assistant Engineer on the Mississippi River Bridge, under the late George S. Morison, Past-President, Am. Soc. C. E., as Chief Engineer, and the late Alfred Noble, Past-President, Am. Soc. C. E., as Resident Engineer. He had special charge of all masonry work and the erection of the west approach viaduct. After the completion of the bridge in the summer of 1892, Mr. Morison put him in charge of the construction of a Memorial Library at Peterborough, N. H. For a short time, also, he was engaged on sewer work at Rochester, N. Y.

In November, 1892, Mr. Angier entered the employ of the Illinois Central Railroad Company as Inspector of Bridges, on both shop and field work, and,

* Memoir prepared by a Committee of the Illinois Section, consisting of E. E. R. Tratman, Assoc. M. Am. Soc. C. E., *Chairman*, Charles L. Strobel, and Arthur L. Webster, Members, Am. Soc. C. E.

later, as Assistant Engineer of Bridges, under the late H. W. Parkhurst, M. Am. Soc. C. E., then Bridge Engineer of that Company. He remained there until January, 1902, when he was appointed Resident Engineer on the Thebes Bridge, his third Mississippi River bridge, under Mr. Noble and Ralph Modjeski, M. Am. Soc. C. E., Chief Engineers. On the completion of the Thebes Bridge, in 1907, he became associated with Mr. Modjeski as his Principal Assistant and, later, was his partner under the firm name of Modjeski and Angier. During the next twenty years he was engaged in the design and erection of several important bridges, his specialty being in connection with difficult foundation work and heavy masonry. In writing of Mr. Angier, Mr. Modjeski states:

"I had a great appreciation for his ability and conscientiousness, which I considered invaluable. His judgment was always characterized by great directness, which often simplified very complex questions. He was of an extremely unassuming nature and a man of very few words."

Mr. Angier suffered a partial paralysis from cerebral lesion in May, 1926, which caused him to retire, although he remained active and retained his interest in engineering work. He was stricken again, at his home in Wheaton, Ill., on Sunday, December 23, 1928, and died on December 29.

In December, 1889, he was married to Mary Powell, of Luling, Tex., who survives him. He leaves also a daughter, M. Estelle Angier, and two sons, Philip Powell Angier, an engineer, of Melville, La., and Robert M. Angier, of Los Angeles, Calif. He was a member of the Western Society of Engineers, and the Chicago Engineers Club.

Mr. Angier was elected an Associate Member of the American Society of Civil Engineers on September 7, 1892, and a Member on September 3, 1902.

LELAND RELLA BALCH, M. Am. Soc. C. E.*

DIED APRIL 18, 1928.

Leland Rella Balch was born in Neillsville, Wis., on March 10, 1883. He was the son of the late Rella W. Balch and Nettie B. (Whipple) Balch, and a descendant in the tenth generation of John Balch, of Somerset, England, who settled in Beverly, Mass., in 1626.

Mr. Balch spent his early life in Neillsville, where he fitted himself for college. He was graduated from the University of Wisconsin with the degree of Bachelor of Science in Civil Engineering in 1905, and received the professional degree of Civil Engineer in 1909.

He spent the summers of 1902-04 as an Assistant with the County Surveyor of Clark County, Wisconsin, and began his professional work with the United States Reclamation Service, first as Engineer's Assistant, then Instrumentman, and, later, as Chief of Party and Resident Engineer in charge of surveys on the Huntley, Sun River, Blackfeet, and Flathead Irrigation Projects in Montana.

From September, 1909, to March, 1910, he took post-graduate work at the University of Wisconsin. In April, 1910, he was appointed an Assistant Engineer with the U. S. Reclamation Service on the Shoshone Project in Wyoming. From November, 1911, to August, 1912, he was engaged as Research Assistant in Hydraulics, at the University of Wisconsin, and spent his time in experimental research and in the preparation of technical *Bulletins* for publication. He prepared several important *Bulletins*, among which may be mentioned: "Tests of Flash Wheels"; "Flow Through Submerged Orifices and Tubes"; and "Hydraulic Curve Resistances". He rendered important service in editorial work in the preparation and revision of various textbooks by D. W. Mead, M. Am. Soc. C. E., including "Water Power Engineering", "Hydrology", and "Contracts, Specifications, and Engineering Relations".

From 1912 until his death, Mr. Balch was with the firm of Daniel W. Mead and Charles V. Seastone, Members, Am. Soc. C. E., Consulting Engineers, of Madison, Wis. With this firm he had responsible charge of the design and construction of many important works, among which may be mentioned the reconstruction of the municipal pumping plants at Madison, Rockford, Ill., Dubuque, Iowa, and Harvard, Ill.; additions to the power plants of the Madison Gas and Electric Company, at Madison; the Mississippi Valley Public Service Company, at Winona, Minn.; the Peninsular Power Company, at Iron River, Mich.; the Eastern Oregon Light and Power Company, at Baker, Ore.; and of the design and construction of the Diesel electric plant of the Peninsular Power Company, at Iron River.

He also participated in investigations and preparation of reports; in the design of numerous water-works, water power, and sanitary works; in divers appraisals and valuations; and also in numerous investigations and reports on power projects and flood-protection works.

* Memoir prepared by Daniel W. Mead, M. Am. Soc. C. E.

Mr. Balch was a man of high ideals, of sterling character, of the highest integrity, and of marked ability. Throughout his career he adhered to the highest of moral and professional standards. A man of pleasing personality, he possessed a genial disposition, and a keen sense of humor, endearing himself to all who knew him.

Mr. Balch was married in Chicago, Ill., on June 15, 1918, to Margaret Elizabeth O'Reilly. Their children are Margaret O'Reilly, Dorothy Jane, James Leland, and Ruth O'Reilly.

He was a member of the American Society of Mechanical Engineers, and the Engineering Society of Wisconsin; an Associate Member of the Wisconsin Utilities Association, and a Member of the Madison Technical Club. He was also a Thirty-second Degree Mason and a member of the Blackhawk Country Club of Madison.

Mr. Balch was elected an Associate Member of the American Society of Civil Engineers on October 5, 1909, and a Member on August 12, 1920.

JOHN WESLEY BALL, M. Am. Soc. C. E.*

DIED JULY 22, 1928.

John Wesley Ball was born at Walton, Ind., on August 8, 1888. He was the elder son of Joseph Ball and Sarah Ann (Johnson) Ball. He was graduated from the Galveston, Ind., High School in 1908, and from 1908 to 1910 attended the Indiana State Normal School. In 1910, he entered the Civil Engineering Department of Purdue University at Lafayette, Ind., from which he was graduated with the degree of Bachelor of Science in Civil Engineering in 1914; he received the degree of Civil Engineer in 1917. During the summer vacations of 1911 and 1912 he worked for the R. C. Bowman Company, of Cleveland, Ohio, and, during the 1913 vacation, as Engineering Assistant for the Cleveland, Cincinnati, Chicago and St. Louis Railway.

After his graduation in June, 1914, Mr. Ball, having passed the required Civil Service examination, was appointed by the late Logan Waller Page, M. Am. Soc. C. E., as Civil Engineering Student of the (then) Office of Public Roads in the United States Department of Agriculture. He worked in the Washington Office until July and was then made Chief of Survey Party on highway location in Sequoia and Yosemite National Parks, reporting to T. Warren Allen, M. Am. Soc. C. E., now Chief of Management in the U. S. Bureau of Public Roads.

A year of this work was succeeded by a transfer, in July, 1915, to Skamania County, Washington, and a promotion to the position of Locating and Designing Engineer for the Bureau on road construction under a $210 000 bond issue in that County. Mr. Ball had sufficiently advanced this work by March, 1916, to permit his transfer to Jackson County, Oregon, to locate 70 miles of proposed highway between Medford, Ore., and Crater Lake National Park. This road survey was an outstanding job for excellence and low cost, and the present road follows closely the line run by Mr. Ball.

On the establishment in 1916 of a District Office at Portland, Ore., by the U. S. Bureau of Public Roads, Mr. Ball was attached to the Staff and during the season from March to November, 1917, he made investigations and reconnaissance surveys and prepared reports on National Forest Highway projects in Idaho, Washington, and Oregon.

On the entrance of the United States into the World War in April, 1917, Mr. Ball applied for admission to the Officers' Training Camp at American Lake, but was rejected on account of a defect in his heart action. In November of that year, however, he became Assistant Engineer for the U. S. Bureau of Public Roads on the paving work in that camp and remained there until February, 1918. During the spring and summer of 1918, he was at the Portland District Office. In October, he took charge of the construction of the Canyonville-Galesville National Forest Highway, in Oregon, which was completed in the fall of 1919. Mr. Ball then went to Lake Crescent,

* Memoir prepared by L. I. Hewes, T. Warren Allen, and J. S. Bright, Members, Am. Soc. C. E.

Washington, as Resident Engineer during the construction of the National Forest Highway there and remained until its completion in April, 1921. These two Forest Highways cost about $600 000 for 20 miles of grading.

When the U. S. Bureau of Public Roads established a Regional Office at San Francisco, Calif., Mr. Ball was selected to take administrative charge of National Forest highway construction in the eleven Western States and Alaska, reporting to L. I. Hewes, M. Am. Soc. C. E., Deputy Chief Engineer. This position he held with distinguished ability. In September, 1927, he was also placed in administrative charge of road construction by the Bureau in the National Parks in the Western States. He remained in charge of all this work in the National Forests and Parks until his death.

Mr. Ball was a man of unusual ability. In college, he was a member of both Phi Beta Kappa and Sigma Xi. His whole professional career was given to public service of a high order. He entered the U. S. Bureau of Public Roads in a minor position on graduation from college and through earnest and continuous application to his work rose step by step until he was filling the highest grade in the service. His technique was excellent and his love for perfect work outstanding. The development of modern standards of location and design in the National Forest and National Park roads owes much to the painstaking care of Mr. Ball in all details. His ability as an administrator constantly developed, and his work was characterized by a quality of humor and intelligence that drew people to him. He was a member of the Mystic Shrine and a Thirty-second Degree Mason.

In 1919, Mr. Ball was married to Ruth Stevenson, of Stevenson, Wash. He is survived by his widow and by a daughter, Elizabeth Ann, and a son, John Wesley Ball, Jr.

Mr. Ball was elected an Associate Member of the American Society of Civil Engineers on July 11, 1921, and a Member on March 15, 1926.

JOHN TONER BARR, M. Am. Soc. C. E.*

DIED JUNE 26, 1925.

John Toner Barr was born in Philadelphia, Pa., on November 25, 1876. He was the son of John C. Barr, of Blairsville, Pa., whose great-grandfather crossed the Allegheny Mountains from Baltimore, Md., long before the Revolution. His mother was Sarah Toner, of Gallipolis, Ohio. Her people came from York County, Pennsylvania, where they had settled during Colonial days.

Mr. Barr attended the public schools of Pittsburgh, Pa., until he was sixteen years of age, when he went to work as Draftsman in the City Engineer's Office. At the end of seven years, in 1898, he gave up this position and became a student in the Western University, now the University of Pittsburgh. He continued his studies at the University, working part of the time as Draftsman for the West Virginia Short Line Railroad Company, now part of the Baltimore and Ohio System, and also for the Allegheny Valley Railroad Company. He was unable, however, to finish his course at the University.

From 1899 to 1902 Mr. Barr worked, successively, as an Engineer for the Allegheny Valley Railroad Company, the Baltimore and Ohio Railroad Company, the Illinois Steel Company, and for the George S. Davison and the Mellon Interests. He designed the plant for the Pittsburgh Reduction Company at St. Louis, Mo., and was also Chief Engineer for the construction of the many miles of interurban electric railways running out of Pittsburgh.

Hard work and close application to the task in hand while fitting himself for his later undertakings began to undermine his health and he was advised to go West for recuperation. While in the West he worked in the office of the City Engineer of Salt Lake City, Utah; for a brief period as Engineer in Thunder Mountain, Idaho; later, at Pitkin, Colo., on mine surveys; and also as Machine Helper on the Oregon Short Line Railroad.

Mr. Barr returned to Pittsburgh in 1904 and became Chief Engineer for his brother, J. Carroll Barr, M. Am. Soc. C. E., and also for the Thomas McNally Company. He remained with them until 1907 when he launched out for himself under the firm name of J. Toner Barr, Civil Engineer, and from 1911 until the time of his death, practiced as J. Toner Barr, Engineer and Contractor. During this time Mr. Barr laid practically all the underground cables for the Bell Telephone Company in the Pittsburgh District.

On the evening of January 6, 1922, while at work in his office, a pistol which he had carried with him for many years for payroll protection accidentally slipped off his desk and was discharged. The bullet struck his right leg, glancing upward, necessitating, five days later, amputation in order to save his life. Mr. Barr met the situation bravely, declaring that he never had made any money with his legs anyway and proceeded to do more business following his accident than he had ever done before. He carried on until June 26, 1925, when he died after an operation for gallstones.

* Memoir prepared by Stewart C. McFarland, Esq., Pittsburgh, Pa.

In 1905, he was married to Anne M. Hogan, of Pittsburgh, who, with four children, Sarah, Rosemary, Jone, and Nellie, survives him. His brother, Mr. J. Carroll Barr, also survives him.

Mr. Barr was a popular member of the Rotary Club of Pittsburgh, holding the office of First Vice-President at the time of his death. The following memorial, delivered before the Club by Past-President and Past District Governor Stewart C. McFarland, who knew him intimately, gives a true picture of Mr. Barr and the esteem in which he was held by his fellows:

"Our friend, our First Vice-President—your friend and my friend—J. Toner Barr, has taken his place in the silent halls of cherished memories.

"Toner's going brings us once again face to face with the Great Inevitable—the final and conclusive fact that no matter how brilliantly a race the white steed of life may run, the black and inexorable steed of death is sure to win.

"When we laid Toner's tired frame in his narrow cell in Calvary last Monday morning we gave back to earth her own. His unconquerable soul and indomitable spirit, his clean life and splendid character, his sense of justice and his loyal devotion to truth, honor, and right are ours forever. This he has bequeathed to us and a richer heritage no man can bequeath to his fellows.

"Toner had a wonderfully discriminating mind and was frankness personified. He never ran away from any situation. He could detect insincerity afar off. He had a keen sense of values. He was seldom if ever misled by the false. He had the hard head of the engineer and the soft heart of the poet. At one time he remarked, 'Take sentiment out of life and all you have left is arithmetic.'

"Toner's home life was a symphony of sentiment. No man ever loved more tenderly his wife and children and no man ever lived in a richer atmosphere of love returned. The happiness of his home life was so complete that at one time he made the statement that he could be happy in Hades.

"J. Toner Barr's seat here to the right of President Jax is now vacant, but in the words of Riley:

> "'We cannot say, and we will not say,
> That he is dead—he is just away;
> With a cheery smile and a wave of the hand,
> He has wandered into an unknown land.'

"To consecrate his memory and to render him a final Rotary service, I propose that we stand for a moment and in that moment pray for the well-being of his soul, and that in our prayer we include his bereaved and broken-hearted widow and children."

At the time of his demise Mr. Barr was an active member of the Chamber of Commerce, the Americus Club (since disbanded), the Engineers Society of Western Pennsylvania, the Historical Society of Western Pennsylvania, and the Duquesne Club of Pittsburgh.

Mr. Barr was elected a Member of the American Society of Civil Engineers on March 4, 1913. He was also a Past-President of the Pittsburgh Section.

GEORGE STRONG BAXTER, M. Am. Soc. C. E.*

DIED JULY 2, 1928.

George Strong Baxter was born in New York, N. Y., on November 21, 1845, the son of George and Anna (Strong) Baxter. His childhood and school days were spent mainly in New York, after which he went to Williams College at Williamstown, Mass., from which he was graduated in 1865. Three years later, he was graduated from the Columbia University School of Engineering.

From 1868 to 1872, Mr. Baxter was employed in land surveying in various parts of the United States. In New York City, he constructed the foundations for the Second and Third Avenue Elevated Lines. He was engaged on the Fourth Avenue Improvement in 1872, became Assistant Engineer, and built the New York Central Viaduct on Park Avenue, north of 96th Street.

In 1879, he became Cashier for the Northern Pacific Railroad Company, and, in 1885, was appointed Assistant Treasurer and sent to St. Paul, Minn. In 1888, he rose to the office of Treasurer and returned to New York. Later, when the Company went into the hands of Receivers, he remained as Treasurer until it was discharged from the Receivership.

In 1895, Mr. Baxter founded the cross-tie business of G. S. Baxter and Company, of New York and Jacksonville, Fla. At first, the firm bought railroad ties in the South and shipped them to New York, but, later, the Company expanded and bought lands in Georgia where it cut its own ties; one sawmill was constructed in Fargo, Ga. In 1900, the Company handled more Southern Pine cross-ties than all other dealers combined.

Owing to some trouble with the Atlantic Coast Line Railroad Company, over shipping ties to tide-water, the Company built a short line east to connect with the Southern Railroad. Mr. Baxter then conceived the idea of a railroad line east to Jacksonville, and west to Valdosta, Ga. The Valdosta terminus made a connection with the Middle West cities and a short route from those cities to the Florida East Coast. The Atlantic, Valdosta and Western Railroad Company was then incorporated with Mr. Baxter as Chairman of the Executive Committee. The various railroad companies fought hard to prevent the admission of the Atlantic, Valdosta and Western Lines to the Union Station in Jacksonville, but they were finally defeated and the railroad was admitted to the Union Terminal rights. Later, the Atlantic, Valdosta and Western Railroad Company considered building west from Valdosta to Birmingham, Ala., and to prevent this, the Southern Railroad Company finally bought the line.

Mr. Baxter was greatly interested in the creosoting of cross-ties, and at one time considered establishing a plant for that purpose. He was also interested in the possibility of bringing tropical hard wood cross-ties into the United States. As the World War cut into the railroad tie business, G. S. Baxter and Company established a shipyard in Jacksonville, on which

* Memoir prepared by Mrs. E. Vogelsang, New York, N. Y.

Mr. Baxter was engaged until 1918 when the partnership was dissolved, and he retired.

In 1874, he was married to Emmeline Carnes Weeks, of New York, the daughter of Emmeline Carnes and Edward Augustus Weeks. She died in 1921. They had five children, of whom three survive: Wyllys Pomeroy Baxter of New York City, George Strong Baxter, Jr., of Westerly, R. I., and Mrs. Erwin Vogelsang, of New York City.

Mr. Baxter was a member of the University and the Mid-Day Clubs. After his return from St. Paul, he resided in New York, and had a summer home at Bellport, L. I. A man of simple tastes and strong affections, he was a most devoted husband and father. His was an unusual intellect and, to the time of his death, he retained his grasp of affairs and interest in life.

Mr. Baxter was elected a Junior of the American Society of Civil Engineers on May 12, 1875, and a Member on May 3, 1876.

WILLARD BEAHAN, M. Am. Soc. C. E.*

DIED FEBRUARY 5, 1928.

Willard Beahan was born in Watkins, N. Y., on January 15, 1854, the son of James and Harriet (Griswold) Beahan. His grandfather was from the north of Ireland and of Scotch-Irish ancestry; his mother was of English descent.

Mr. Beahan prepared for college at Starkey Seminary, Starkey, N. Y., and was graduated in 1873. He entered Cornell University, Ithaca, N. Y., in 1874, from which he was graduated in 1878 with the degree of Bachelor in Civil Engineering. He took a prominent part in class and college activities, playing on his class football team and in his Senior year being elected Class Historian and Chief Engineer. After graduation he was elected to Sigma XI and Tau Beta Pi, honorary fraternities. He also became Life Secretary of his Class, and at the time of his death was preparing for the 50-year re-union.

After his graduation from Cornell University in 1878 he entered the service of the United States Corps of Engineers as Assistant Instrumentman and, later, as Computer on the Mississippi River Survey. In 1880, he was employed by the Gould Lines in the Southwest as a Rodman on the Texas and Pacific Railway. He was advanced to the position of Division Engineer in 1881 and then became Division Engineer on the Fort Worth and Denver City Railway where he was in charge of location and construction.

After an interval of one year, from March, 1883, to March, 1884, during which time he was employed as a Transitman on the Lehigh Valley Railroad, Mr. Beahan returned to the Southwest as Division Engineer on the Missouri Pacific Railway, where he remained until February, 1889, engaged in location, construction, and maintenance. He then went to South America with the North and South American Construction Company as Superintendent of Construction and Bridge Engineer of the Chilean Railway. Before the end of the year a revolution interfered with the progress of this work and Mr. Beahan crossed the Andes on a mule, made his way to Buenos Aires, Argentina, and returned to the United States *viâ* Europe.

During 1891, he was Superintendent of Streets of St. Louis, Mo. In after years, Mr. Beahan was accustomed to refer to this engagement as the time when he was political "boss" of St. Louis.

From November, 1891, to December, 1896, he was with the Anderson and Barr Company in various capacities. As President of the Anderson and Barr Clay Company, he developed a large and important business in the manufacture of clay products, in Streator, Ill. In 1896 and 1897 he was Lecturer on Railroad Location at Cornell and Leland Stanford, Jr., Universities.

From August, 1897, to May, 1898, Mr. Beahan was Superintendent of Construction on the Cascade Tunnel on the Great Northern Railway. From September, 1898, to September, 1900, he was again with the Lehigh Valley Railroad Company as Engineer of Maintenance, following which, he served

* Memoir prepared by George H. Tinker, M. Am. Soc. C. E.

as Division Engineer on the Chicago and Northwestern Railway until March, 1905.

From 1905 to 1924, Mr. Beahan was First Assistant Engineer on the staff of the Chief Engineer of the New York Central Railroad Company at Cleveland, Ohio. This long engagement ended with his retirement under the age and pension rules. Immediately thereafter, he entered the service of the "Nickel Plate Road" (New York, Chicago, and St. Louis Railroad Company) as Special Engineer in charge of relocation surveys. After apparently recovering from the effects of a serious operation, he continued to do consulting work for the Erie Railroad Company, but in the latter part of 1927 his health began to fail. He died on February 5, 1928.

In 1892, he was married to Bessie B. DeWitt, who survives him. He is also survived by two brothers.

Mr. Beahan's interest in Cornell University continued throughout his life. He was a member of the Cornell Society of Civil Engineers of New York and was one of the leaders in organizing the Northeastern Ohio Cornell Association. He was elected Alumni Trustee of Cornell University in 1900, and, again, in 1909.

In addition to Cornell and Leland Stanford, Jr., Universities, he lectured before many educational and technical societies. His interest in education and the guidance of young engineers never lessened. He was a member of the Board of Managers of the Cleveland Young Men's Christian Association and, for years, served as Chairman of the Committee on Technical Education of the Cleveland Engineering Society. His many talks to young engineers and students on the subject of Human Engineering were illustrated copiously by incidents from his own long and notable career.

During his long residence in Cleveland, Mr. Beahan became identified with the civic and professional life of the community. He was a member of the Congregational Church and was especially a leader and adviser of young men. He was a member of the Cleveland Engineering Society (President, 1908), Board of Civic League of Cleveland, Cleveland Chamber of Commerce, a Director of the Bureau of Municipal Research, and a member of the American Railway Engineering Association. He was also the first President of the Cleveland Section of the Society.

He was the author of "The Field Practice of Railway Location", and was also a contributor to the publications of the Society.

Mr. Beahan was elected a Member of the American Society of Civil Engineers on April 3, 1889, and served as a Director from 1919 to 1921.

HEZEKIAH BISSELL, M. Am. Soc. C. E.*

DIED JUNE 23, 1928.

Hezekiah Bissell was born at East Windsor (Scantic), Conn., on February 7, 1835, the son of John Bissell and Elizabeth (Thompson) Bissell. He was a direct descendant of John Bissell who settled in Windsor, Conn., in 1639, and a year later removed to the east side of the Connecticut River.

Mr. Bissell attended the schools of East Windsor and Williston Academy at Easthampton, Mass., where he prepared for college. He entered Sheffield School at Yale University in 1857, but three months before his graduation in 1861 he left school and enlisted at Hartford, Conn., in Company A of the First Regiment, Connecticut Volunteers. He was one of the first to answer the call for the three months' volunteer service, enlisting on April 18, 1861, serving his term of ninety days, and being mustered out on July 31, 1861. On August 23, 1862, he re-enlisted in Company G, 25th Connecticut Volunteers, and was chosen First Lieutenant of that Company, which was largely made up of volunteers from his home town of East Windsor and the neighboring towns of South Windsor and Glastonbury, Conn.

This regiment served with General Banks in the Louisiana Campaign, and was engaged in the siege of Port Hudson and the Battle of Bayou Teche. Mr. Bissell's second service being a nine months' enlistment, he served his full time, and was mustered out on August 26, 1863, so that his complete service was for nearly sixteen months.

With the other students of Yale who left college to enter the Army in 1861, he was given the degree of Bachelor of Philosophy in 1881.

As an example of Mr. Bissell's oneness of purpose the following is taken from his personal memoirs: "When a pupil in Williston he was asked by Mr. J. M. Stoughton, an old friend of his father, what he intended to do, and his reply was, 'I am going to help build the Pacific Railroad; then go to South America to build more railroads; then for my real life work be Engineer of some first-class railroad in the United States; later, I will invent something to give me all the money I want to enjoy life'." And in his personal memoirs written after his retirement from all business, Mr. Bissell adds this statement, "I have done all but the last of these."

Soon after his discharge from the Army Mr. Bissell went to New York, N. Y., and at the office of the Union Pacific Railroad Company in that city, met the late Thomas C. Durant, F. Am. Soc. C. E., President, and Peter A. Dey, Chief Engineer, of the proposed railroad. As a result of this interview he was appointed Second Assistant Engineer by Gen. John A. Dix and, on April 1, 1864, reported at that Company's office at Omaha, Nebr. From this date until its completion on May 10, 1869, he was engaged on some part of the great undertaking.

His positions on this work were that of Second Assistant Engineer, First Assistant Engineer, Division Engineer, and Resident Engineer, but in the

* Memoir prepared by Lester W. Tucker, M. Am. Soc. C. E., and Walter H. Norris, Assoc. M. Am. Soc. C. E.

latter part of his service much of his time was spent in special assignments on various important undertakings. As Locating Engineer, Mr. Bissell located and built much of the original line from Laramie, Wyo., to west of Green River, and he was one of the party who ran the preliminary trial lines from the south end of Great Salt Lake westerly into what was then called the Great American Desert, and, with this party, endured great hardship. He was the only Engineer who stayed with the undertaking from the beginning of the work to its completion, a period of more than five and one-half years. In his own words, he had "staked out the first culvert at Omaha and was present when the gold spike was driven at Promontory Point."

In 1901, Mr. Bissell, while on a trip to the West, stopped in Omaha and met Mr. Burt, then President of the Union Pacific Railroad Company. Mr. Burt invited him to ride over the road with which he had been so intimately connected thirty-five years before, furnishing him a car and the Division Engineer as escort, and arranging for the entire trip to be made by daylight.

Again, in 1919, on the occasion of the Fiftieth Anniversary of the driving of the gold spike, Mr. Kennedy and Mr. Bissell were guests of the Railroad Company, and were the only representatives of the original engineering officials.

Mr. Bissell's work on the Union Pacific Railroad closed in June, 1869, and he returned East, but had been there only a short time when he was asked by Dr. Durant, the former President of the Union Pacific Lines, to undertake the location of the Adirondack Railroad then proposed to be run from Ogdensburg, *via* Canton, to Saratoga, N. Y. Surveys on this line were started late in August, 1869, and from that time until the fall of 1870 the work of surveying and grading was carried forward; but during 1871, money being scarce, little was done, and the work on this line was closed.

Early in February, 1872, he received an offer to go to South America on the roads then being built and projected by Mr. Henry Meiggs for the Peruvian Government. He sailed from New York on May 15, 1872, and after a very stormy passage to Colon (then called Aspinwall), and a slow trip down the West Coast, he finally landed at Mollendo, the Port of Arequipa, Peru.

His work was that of locating and building the railroad from Juliaca, northerly up the Pucara River for a distance of about 100 miles, this being on what was projected as the main line from Puno, a port on Lake Titicaca, to Cuzco, the ancient Inca Capital of Peru. This section, being northerly of Lake Titicaca, was located on the backbone of the Andes Mountains, and was from 12 000 to 14 000 ft. in elevation for its entire length.

The 100 miles were located and graded complete and ready for track and Mr. Bissell was ordered to establish his camps and get ready for the location and grading of the remainder of the line into Cuzco when he received sudden orders to stop all work and leave the country at once. A revolution of great intensity was under way in the northern districts of Peru, and all Americans had to flee.

Mr. Bissell was engaged for two years on this work, the entire period having been spent at these high altitudes with few, if any, white men, and at a time

when it was necessary to live as the natives lived; he always thought that his thin hair and beard were one of the results of his mode of living during these two years.

Returning to his home from this work early in 1875, he received an offer to go with the Cleveland, Cincinnati, Chicago and St. Louis Railway Company and for the next three years was with that road in various engineering capacities, revising locations, examining bridges, and constructing cut-offs from Cincinnati, Ohio, to the north and west.

On February 1, 1878, Mr. Bissell was appointed Master of Maintenance of Way of the Eastern Railroad Company extending from Boston, Mass., to Portland, Me., with headquarters at Salem, Mass. The Eastern then had 270 miles of road. In 1884, it was taken over by the Boston and Maine Railroad Company and Mr. Bissell was made Chief Engineer of the Boston and Maine Lines, with headquarters at Boston. In 1886, the Boston and Lowell Lines were taken over by the Boston and Maine, and, in 1900, the Fitchburg Lines came under the same management, so that Mr. Bissell became the Chief Engineer of one of the largest railroads in the East with a mileage of 2 300.

While acting as Chief Engineer of the Boston and Maine System, Mr. Bissell had charge of the rebuilding of much of the roadbed, many of the bridges, and the greater number of the railroad stations on that line. He relocated many miles of the first road, double-tracked much of the original Eastern and Boston and Maine Lines, and reduced grades and curves, eliminated grade crossings, and built many new yards, terminals, wharves, and docks.

On June 30, 1909, Mr. Bissell resigned his position as Chief Engineer after having served thirty-one years and, in his announcement, President Tuttle retired him with the title of Consulting Engineer and paid him an exceptional compliment of appreciation for his long and faithful service. The following year he went to California where he made his home in Altadena.

Mr. Bissell was a man of rare intelligence, great integrity, and oneness of purpose. He held the respect and loyalty of all those with whom he came in contact whether they were the men of his own organization or those of the contractors and builders with whom he was so closely associated. His directness of purpose, absolute honesty, and ability to see the other fellow's point of view made engineers and contractors alike respect him and his decisions. He was especially kind and helpful to the young engineers of his organization.

Mr. Bissell was twice married; his first wife was Alice Hughes, of St. Thomas, Ont., Canada, to whom he was married in 1875 and by whom he had three sons, all living. His second wife, Segrid Johansen, to whom he was married in 1910, died several years later.

Mr. Bissell was a member of the Yale Engineering Association, Boston Society of Civil Engineers, Southern California Association of Engineers, Engineers' and Architects' Association, and the Yale Alumni Association. During his long residence in Boston he was a member of many of the leading social and literary clubs of that city, but withdrew from most of these when

he went to California. In his early manhood he was a member of the Congregational Church, but after his marriage he united with the Protestant Episcopal Church.

Mr. Bissell was elected a Member of the American Society of Civil Engineers on September 15, 1869, and at the time of his death was the third oldest member of the Society, being exceeded only by the late Cady Staley, M. Am. Soc. C. E., who was elected on March 17, 1869, and by Clemens Herschel, Past-President and Hon. M. Am. Soc. C. E., who was elected on September 21, 1869. Mr. Bissell served as a Director of the Society from 1905 to 1907.

WILLIAM HERBERT BIXBY, M. Am. Soc. C. E.*

DIED SEPTEMBER 29, 1928.

William Herbert Bixby, the son of Clark Smith and Elizabeth (Clark) Bixby, was born in Charlestown, Mass., on December 27, 1849. He was descended from Colonial stock, his forebears having served in the Colonial and Revolutionary Wars. He attended the public schools of Brookline and Cambridge, Mass., and then entered the Massachusetts Institute of Technology. After one year at the latter institution he was admitted in 1869 to the United States Military Academy at West Point, N. Y., from which he was graduated in 1873, at the head of his class, and commissioned a Second Lieutenant in the Corps of Engineers, U. S. Army.

Lieutenant Bixby's first assignment was at the Engineer School of Application, Willets Point (now Fort Totten), N. Y., from 1873 to 1875. In 1875 he was assigned to duty at the United States Military Academy where he served as Instructor and Assistant Professor of Civil and Military Engineering until 1879. In 1879 he was sent on duty to Europe where he remained until 1882, attending the Ecole Nationale des Ponts et Chaussées of France, from which he was graduated with honor in 1881. During his tour of duty abroad, he also made numerous investigations of European practice in civil and military engineering. He received the decoration of Chevalier of the Legion of Honor of France for assisting at the maneuvers of the French Army in 1880.

Returning to the United States, he was placed on duty with the Engineer Battalion at Willets Point, where he remained until 1884. By this time he had reached the grade of Captain in the Corps of Engineers, and, from 1884 to 1910, he served as Captain, Major, Lieutenant-Colonel, and Colonel, on river and harbor or lighthouse duty at Wilmington, N. C., Newport, R. I., and Philadelphia, Pa.; at Cincinnati, Ohio, in charge of the Ohio River System; on the Great Lakes at Detroit, Mich., and Chicago, Ill.; and at St. Louis, Mo., as President of the Mississippi River Commission. In 1910, he was appointed Brigadier-General, Chief of Engineers, and served in this capacity until his retirement from active service in 1913.

On the entrance of the United States into the World War in 1917, General Bixby was recalled to active duty, and served until 1919 supervising the work of river and harbor improvement on the Mississippi River and its tributaries, in order to release officers on the active list for war service.

The foregoing outline of General Bixby's service gives only a general indication of the character and variety of his duties during his long and active life. In addition, he was a member of many Official Boards, which greatly extended the scope of his activities; and it is no exaggeration to state that there are few of the manifold activities of the Corps of Engineers to which he did not contribute during his active service of more than forty years. Possessed of a brilliant mind, a tireless energy, and a seemingly unlimited

* Memoir prepared by Herbert Deakyne, M. Am. Soc. C. E.

capacity for study and research, he acquired an almost inexhaustible fund of knowledge of civil engineering works, on which he drew in the consideration of the multifarious problems that presented themselves during his career.

One important board of which General Bixby was a member was appointed by the Chief of Engineers in January, 1894, pursuant to instructions of the Secretary of War, the late Hon. Daniel S. Lamont, to determine the maximum practicable span for suspension bridges. The other members were the late Gen. (then Major) Charles W. Raymond, U. S. Army, M. Am. Soc. C. E., and Col. (then First Lieutenant, Corps of Engineers) Edward Burr, U. S. Army, (*Retired*), M. Am. Soc. C. E. The appointment of the Board was due to strenuous efforts made by interested parties to secure the passage by Congress of a bill authorizing the construction of a bridge across the Hudson River at New York, N. Y., and specifically authorizing a pier about mid-channel. The proponents of the bridge maintained that it was a crying necessity and could not be built as a single span or without a channel pier. The Chief of Engineers strongly opposed the deliberate placing of any such obstruction as a bridge pier in mid-channel. Mr. Lamont directed the Chief of Engineers to convene a Board of Officers to investigate and report its conclusions as to the maximum length of span practicable for suspension bridges and consistent with an amount of traffic probably sufficient to warrant the expense of construction.

The members of the Board, as is customary in such cases, were given this work in addition to their regular duties. General (then Captain) Bixby, at this time, was on duty in charge of the U. S. Engineer Office at Newport, R. I. The Board, after a study of the task before it, assigned to each member particular lines of investigation, in addition to a general study of the problem as a whole. Among other things, the subject of wind pressures was assigned to Captain Bixby. The result of his investigations is printed as an Appendix to the Board's report and is characteristic of his efficiency, thoroughness, and almost infinite capacity for detail. Captain Bixby's compilation and discussion was unique in its field and filled a need long felt by engineers of that day, particularly in bridge design. The report of the Board was printed not only as an official document but also in the engineering press. *Engineering News*, in addition to printing the report, stated in an editorial* that it "forms one of the most valuable and instructive engineering investigations of the day".

The report set the accepted possibilities as to length of bridge spans far in advance of the practice then current, and preserved the Hudson River and doubtless many other important waterways from serious obstruction by bridge piers. The conclusions of the Board have been justified by the construction of the Fort Washington Bridge across the Hudson with a span of 3 500 ft., which is still well within the maximum given by the Board of about 4 300 ft.

In 1909, General (then Colonel) Bixby was selected as Advisory Engineer to the National Waterways Commission, a body created by the Act of Con-

* *Engineering News*, November 22, 1894.

gress of March 2, 1909, which Commission he accompanied to Europe. His detailed knowledge of works for the improvement of navigation both in Europe and the United States made him of great value to the Commission.

The wreck of the battleship *Maine* was removed from Havana Harbor while General Bixby was Chief of Engineers. He was deeply interested in this work, and gave it a great deal of personal attention, until the success of the undertaking was assured.

General Bixby was as active physically as mentally. As a young man he was an expert skater and, when on duty in France, took occasion during one winter to visit Holland in the hope of acquiring some of the refinements in the art. It developed, however, that his ability as a skater was already so great that he became a teacher rather than a student of the Hollanders. He found time while abroad for mountain climbing and ascended the Matterhorn with two guides. On another occasion he left Zermatt, Switzerland, about noon with one guide and ascended the Breithorn. Returning to the village about noon the next day he met a party starting out to make the ascent. They urged him so strongly to accompany them that he consented and re-ascended this mountain, getting back to the village about 4:00 P. M. of the following day, after more than 48 hours of continuous mountain climbing.

General Bixby was always intensely interested in new devices. As an Instructor at the United States Military Academy, during the period from 1875 to 1879, he foresaw the possibilities of the typewriter, then a crude machine compared to present-day models, and endeavored to secure its introduction to take the place of the laborious and time-consuming pen and ink method by which Army correspondence was then conducted.

He also took a great interest in the improvement and simplification of administrative procedure and was largely responsible for a number of methods now in use by the Corps of Engineers. One of these was the standard specifications for dredging, which he initiated and which are now in use throughout the Engineer Department for all dredging contracts. His interest extended to almost every field of scientific endeavor. The number of scientific publications to which he subscribed was so great that it was his custom to rent a private office to which all such periodicals were sent and where his professional library was kept in order to avoid overtaxing the facilities of the office assigned him for the transaction of public business.

General Bixby was a man of simple tastes; there was nothing of the martinet about him. He loved the discussion of matters of professional interest, and his tremendous fund of knowledge on military and civil engineering, and other scientific subjects, was a source of astonishment to those to whom it was displayed. He was one of the most generous and kind-hearted of men, always thoughtful of the feelings and comfort of others, and most considerate in his dealings with them. His character was unimpeachable, and he leaves a record of service to his country unsullied by the slightest spot of selfish interest.

He belongs to that remarkable group of engineers, now nearly all gone, whose active lives spanned the period between the close of the Civil War and

the outbreak of the World War; an era in which the advance in scientific knowledge and engineering achievement was so great, that some future historian of scientific progress may well describe it as the "Golden Age" of Science.

General Bixby's intense interest in science and engineering continued undiminished throughout his active life and to the end. In spite of advancing years, he was vigorous both in mind and body until stricken with his last illness when death stopped forever the strivings of his restless intelligence. He died in Washington, D. C., and after a simple service at his home, his ashes were placed in the Arlington National Cemetery.

He was married on December 27, 1893, to Mrs. H. M. Jones, of Philadelphia, the widow of Lieut. H. M. Jones, 4th U. S. Artillery. Mrs. Bixby survives him.

General Bixby was a member of the French Society of Civil Engineers, the British Institution of Civil Engineers, the American Society of Mechanical Engineers, the Institute of Construction Engineers, the Academy of Political and Social Science, and other societies of Political Economy, the American Association of Port Authorities, and the International Association of Navigation Congresses. He was President of the International Congress of Navigation, held at Philadelphia in 1912; Chairman of the Engineering Section of the Pan-American Scientific Congress at Washington, D. C., in 1915; President of the American Society for Testing Materials in 1917-18; and President, in 1918-19, of the American Society of Terminal Engineers.

He also held memberships in the Society of American Military Engineers, the U. S. Military Service Institution, the U. S. Infantry Association, the U. S. Cavalry Association, the U. S. Naval Institute, the American and British Associations for the Advancement of Science, the Mathematical Society of America, and the American Academy of Arts and Sciences.

General Bixby was elected a Member of the American Society of Civil Engineers on April 5, 1882.

WILLIAM TITCOMB BLUNT, M. Am. Soc. C. E.*

DIED MARCH 3, 1928.

William Titcomb Blunt, the only son of Aaron Drew and Helen Maria (Titcomb) Blunt, was born at Norridgewock, Me., on January 10, 1855. He received his early education in the Grammar School and Eaton Family School of his native town, entering the Massachusetts Institute of Technology in 1870, from which he was graduated in 1874 with the degree of Bachelor of Science in Civil Engineering.

During the summer vacations of 1872 and 1873, Mr. Blunt was employed by the City Engineer of Somerville, Mass., and by others, on topographic and hydrographic surveys. Following his graduation in 1874, he secured a position with the U. S. Coast and Geodetic Survey as Observer with a party making a survey of the mouths of the Mississippi River.

From 1875 to 1879, Mr. Blunt served as Clerk and Assistant Engineer with the Harbor Commission of Massachusetts on triangulation, topography, hydrography, and mapping. From 1879 to 1883, he was engaged, for the Mississippi River Commission, on triangulation, topography, and hydrography along the lower sections of the Mississippi River. In 1883 he became associated with Mr. John Eisenmann under the firm name of Eisenmann and Blunt, at Cleveland, Ohio, in the practice of architecture and engineering.

From 1886 to 1890, Mr. Blunt was employed under the U. S. Engineer Office at Cleveland, as Inspector on the maintenance and improvement of the harbors along the south shore of Lake Erie. During the last six months of 1890 he was Principal Assistant Engineer in Charge of Surveys for the Sanitary District of Chicago, Ill., but resigned with the Chief Engineer, the late Lyman E. Cooley, M. Am. Soc. C. E., on account of their objections to the way the work was being conducted by the Board of Trustees.

For a time Mr. Blunt was busy in the insurance business, but the work was not congenial to him, and he soon returned to engineering. From 1892 to 1906, he was continuously employed as U. S. Assistant Engineer in the maintenance and improvement of harbors and the location and construction of lighthouses along the south shore of Lake Erie. This work was under the U. S. Engineer Office at Cleveland, but in immediate charge of the Sub-Office at Toledo, Ohio, where Mr. Blunt resided during this period.

From 1906 to 1907, he served as Superintendent of construction with George H. Breymann and Brothers, Contractors, of Boston, Mass., on river and harbor work, especially on subaqueous rock excavation, and, from 1908 to 1910, he was engaged on similar work for the U. S. Government at the Pacific entrance of the Panama Canal. In connection with this work he designed a rock crusher that was said to be superior to the Lobnitz crusher, which was also in use.

During the following two years Mr. Blunt remained on his farm in Ohio, but in November, 1912, he became General Manager for the Shirley-Skeldon Dredging and Submarine Contracting Company, of Toledo, with which he remained until May, 1914.

* Memoir prepared by the following Committee of the Illinois Section: John W. Woermann, Clifford C. Muhs, and Edward J. Fucik, Members, Am. Soc. C. E.

From June, 1914, until his retirement from the Federal Civil Service, in January, 1925, on account of having reached the age limit prescribed by law, he was employed as Assistant Engineer, under the U. S. Engineer Office at Chicago on the maintenance and improvement of the harbors at the south end of Lake Michigan. This work included dredging operations in eight rivers and harbors, rubble mound and concrete caisson breakwaters at Chicago Harbor and Indiana Harbor, Ind., and the construction of a concrete superstructure on the old timber breakwater at Calumet Harbor, Ill. For a short period during the World War he was Acting District Engineer in charge of the office.

After his retirement from Government Service, Mr. Blunt was engaged by the Universal Portland Cement Company as Consulting Engineer on the construction of a new harbor at Buffington, Ind., including 1 300 ft. of rubble-mound breakwater. Following this, he was retained by the D. P. Davis Company, Realtors, as Staff Engineer in charge of extensive dredging operations at St. Augustine, Fla., on the development of a new seaside sub-division.

After the collapse of the Florida land boom in 1927 he returned to Chicago, and became affiliated with the Division of Waterways of the State of Illinois. During the summer and fall he was in charge of the repair of levees along the Mississippi River between St. Louis, Mo., and Cairo, Ill., and during the winter he was engaged in miscellaneous office work in Chicago.

Mr. Blunt was extremely thorough and careful in his work, and remained active and energetic until eight days before his death, which was due to pneumonia following a cold. He made loyal friends wherever he was located, and enjoyed nothing more than an evening at cards or story-telling. He has been characterized by one of his friends as "a rare raconteur tempered by the fire of a crusader; a *bon vivant* tempered by a fine æstheticism; a true friend, and a fair opponent."

He was twice married, first, in 1874, to Frances Sydney Gibbs, of Toledo, Ohio, who died in 1907, and then, in 1909, to Helen Elizabeth Norton, who survives him.

Mr. Blunt was a Royal Arch Mason and a Charter Member of the Royal Arcanum of Cleveland. He was a member of the American Association of Engineers and also of the Western Society of Engineers. At the monthly luncheons of the M. I. T. Club of Chicago, he was usually the oldest graduate present.

Mr. Blunt was elected a Member of the American Society of Civil Engineers on October 5, 1898.

GEORGE HERBERT BOYNTON, M. Am. Soc. C. E.*

DIED APRIL 20, 1928.

George Herbert Boynton was born at Meredith Village, N. H., on September 3, 1852, the son of Emory Warren Boynton and Mary (Cram) Boynton. His early boyhood was spent in the neighborhood of Watertown, Mass. He began his education at the age of three, in Lawrence, Mass., while accompanying his brother to the little "old red brick school house", which was still standing a few years ago. He, thus early in life, showed himself very persistent in any thing he undertook, consequently, his father found it necessary, at times, to carry him to school through stormy weather.

When Mr. Boynton was six or seven years old, he began his business life by delivering papers. About this time his mother died. After a few years his father was married to Eliza Hilton, who was an exceptionally fine woman, and no boy ever could have been a more devoted son than Mr. Boynton was to her until she died in her seventies.

In 1866 the family moved to Rock Island, Ill., where he attended the public schools. Soon thereafter he went to work on the Government Island where he served as Time-Keeper for a short period. In order to be early, so that he might set an example for the many men employed there by "Uncle Sam", he was obliged to get up before daylight, as he had to cross the Mississippi River and walk several miles to the head of the Island. At one time, owing to great damage by storms to the wagon bridge, he found it necessary to walk the railroad bridge. When one stops to consider the darkness, the snow, and the ice, one may realize, somewhat, the energy and ambition of a "young man" not yet seventeen. He was never late and never complained. His method of "carrying on" then, was typical of his whole after life.

Mr. Boynton next engaged in office work at the Rock Island Arsenal, and was shortly appointed Assistant to the Chief Draftsman, Mr. Otto Gronen. While thus occupied, Lieutenant Shaler became so much interested in him, that he gave him a liberal course in mathematics. It was largely due to the interest shown by his two superiors, that Mr. Boynton decided to make an effort to continue his education along engineering lines. With the encouragement and help from his family and a good friend or two, whom he later repaid, he succeeded in going to Germany in 1872, where he entered the Polytechnic School in Chemnitz, Saxony. He at once took up the strenuous task of beginning the study of German and his engineering course in that language, at the same time. He came out with flying colors in both studies and was graduated in 1875. Later in life he found much pleasure in German literature.

When he first entered the Polytechnic School he met a Norwegian of about his own age. They were attracted to each other in the unfamiliar land in which they met. At any rate, a friendship was formed which was still loyal and touching at the time of Mr. Boynton's death. "George" spent his vacations in Norway at the home of "Jens" Skougaard, and "Jens" afterward visited his friend's home in the United States many times.

* Memoir prepared by H. W. Zeug, Secretary, Northern Gravel Co., Muscatine, Iowa.

After three years of study, Mr. Boynton returned to Rock Island, ready and eager to go to work. For a time he carried on a private practice. In 1877, he was made City Engineer. This work, apparently governed largely by politics, was not entirely to his liking, and at the expiration of his term of office, in 1878, he went to the Chicago, Rock Island, and Pacific Railway Company, expecting temporary employment only, but evidently he was successful, as more and more important duties were given him, and he remained with that Company for twenty-nine years. During the first summer, he was appointed Assistant Engineer on a survey of the Mississippi River between Keokuk, Iowa, and Grafton, Ill. In the fall of 1879, he was appointed Assistant Engineer with headquarters in Davenport, Iowa. He was there for twenty years, being then transferred to the Chicago Office, where he remained until his resignation in 1907.

In 1908 he returned to Davenport and entered the general contracting business, being engaged in the construction of bridges and railroads, until 1914, when he organized the Northern Gravel Company, of Muscatine, Iowa, where he went to live a few years later. He was elected President of the Company, and held that office until he died on April 20, 1928.

Before the World War was declared, Mr. Boynton was made a member of the Naval Construction Board of the United States, organized for industrial preparedness, Mr. Thomas A. Edison being Chairman. Mr. Boynton gave his services to his country as one of the State Directors, as long as the Board continued to function.

In 1884, he had a prolonged and serious attack of typhoid fever. The first thing to make an impression on him when he came back to his normal conscious life was a notification that he had been elected to membership in the Society. It was dated September 3, 1884, which happened to be his thirty-second birthday. On November 30, 1918, thirty-four years later, he received another official notice from the Society stating that, "A Corporate member who has paid dues twenty-five years, and is seventy years old, is exempt from further payment of dues", and it further stated that he had been made a "Life Member of the Society". While for various good reasons he never took an active part in the Society's work, he was much interested in its activities and kept in touch with them, at the same time enjoying the friendship and acquaintance of many of his fellow members.

Mr. Boynton possessed a lovable personality and although he was rather quiet and reserved, his kindly interest in people, especially in younger men, seemed to invite and inspire confidence. With his high ideals of life and integrity, his own daily life seemed to be a helpful influence to many with whom he came in contact.

During the years he lived in Muscatine, he made warm friends, both in his social and business life. He was a dependable and valuable member of the Board of Directors of the Hershey State Bank, and was often called on for counsel and advice in the affairs of the city. He was a member of the Geneva Golf and Country Club and, when living in Davenport, was at one time President of the Outing Club. Mr. Boynton was a great home lover, and was fortunate in having a comfortable old-fashioned house on the bluff

overlooking the Mississippi River. He was never so happy as when at home with his wife and books by his side, having been an inveterate reader from childhood. While fond of music, having inherited a good voice and ear from his mother, walking and reading were his main recreations. Consequently, it was an unusually great sorrow when in September, 1927, an acute attack of glaucoma demanded an immediate operation, which allowed no more reading or writing. For years one of his favorite maxims had been, "there is no use kicking against the pricks", and there was little complaining; on the contrary, a feeling of thankfulness that he had been spared pain and that being in apparently excellent health, he could do everything as usual, except read and write.

It was a great shock to every one when, with no warning whatever, he was suddenly stricken with angina pectoris on January 13, 1928. He suffered great pain for a few days, which was followed by exhaustion. Continued rest and relaxation brought back his strength in no small degree, and for many weeks out of the fourteen in which he did not leave his room, he kept in constant touch with his business and the affairs of the Nation.

It may be of interest and appropriate to quote here an article that appeared in the *Muscatine Journal*, as it seemed to meet with the approval of many. It was written by a beloved and admired friend, a rather widely known historian and writer. The heading was:

"A Certain Man

"Men there are who, when taken from the world, leave it poor indeed. Such a man was Mr. George H. Boynton, of Muscatine. Mr. Boynton had a mind of technical bent. Technical things attracted him. A Civil Engineer, he so followed his calling as to be honored with membership in the American Society of Civil Engineers, later being made an Honorary or Life Member.

"But it is not of Mr. Boynton's practical aptitudes that the writer would speak. He had qualities far rarer. These caused him to be known as 'of the old school', a man of courtesy, of chivalry, a gentleman. Old Thomas Decker hesitated not to describe 'The best of men that e'er wore earth about him' as simply 'the first true gentleman'. Distinguished for courtesy, for kindliness, Mr. Boynton did not lack firmness; he was steadfast, he stood four-square to any winds that blew.

"A man's education, his training may or may not single him out. Mr. Boynton's training singled him out notably. In the main, he gained it in Germany. It was thorough—that was German. It was scrupulous—that was New England. Mr. Boynton's scrupulous thoroughness marked him from first to last. The particular thing that was just—this to him—was the vital thing.

"Kindliness evidenced in courtesy; courage evidenced in steadfastness; honor evidenced in scrupulous regard for what was just—such was Mr. Boynton's life."

On December 31, 1889, Mr. Boynton was married, in Davenport, to Frances E. Cutter, of Rock Island, who survives him.

The funeral service at his home in Muscatine was simple and modest—as typical of the man, and as he would have wished it to be. The concluding sentence of this service was most appropriate and covered his whole life. It was: "And so: We leave our brother—our friend—the husband who was still the lover".

Mr. Boynton was elected a Member of the American Society of Civil Engineers on September 3, 1884.

GEORGE HAMPTON BREMNER, M. Am. Soc. C. E.*

DIED APRIL 3, 1927.

George Hampton Bremner was born at Marshalltown, Iowa, on December 16, 1861. His father, William Bremner, came to America from Eastern Scotland in early childhood with his parents who settled in New England. After preparing for both law and engineering, William Bremner was actively engaged for a number of years in railway construction, chiefly in Connecticut. Joining the westward movement, he migrated to the State of Iowa and there was married to Catherine Hampton, of Iowa City, who was of English stock and a direct descendant of Governor Chittenden of Vermont. In 1856, they settled in Marshalltown, Iowa, where they reared their family. William Bremner served in the State Legislature in the early Sixties and practiced his two professions for many years, serving as City Engineer and as County Surveyor at Marshalltown until the end of his active professional life.

As a boy, George Hampton Bremner became familiar with the technique of instruments and practical methods of construction by helping his father. Railroads presented an attraction for him, however, and he spent his vacations, while attending the University of Iowa at Iowa City, Iowa, as a Rodman with Middle Western railway companies. After his graduation from the University in 1883, he worked with his father for two years, and then entered the employ of the Chicago, Burlington and Quincy Railroad Company in 1885, as a Division Engineer at Red Oak, Iowa.

From 1889 to 1890 Mr. Bremner was in charge of a locating party on the St. Louis, Keokuk, and Northwestern Railroad (a subsidiary of the Burlington), locating a line between Old Monroe and St. Louis, Mo., which section includes the crossing of the Missouri River near Bellefontaine, Mo. At the conclusion of this work he returned to the Chicago, Burlington and Quincy Railroad Company, and was placed in charge of the maintenance of lines in Northern Illinois with headquarters in Chicago. In 1898 the Burlington undertook its first track elevation (grade separation) at Chicago and Mr. Bremner was placed in charge. On this work he made an enviable record, taking special pride in the speed, economy, and quality of this pioneer construction, while handling it with a minimum of interference with train operation. In 1904, he was transferred to the Operating Department as Engineer, Maintenance of Way, of the Illinois Lines. He returned to the Engineering Department in 1908 as District Engineer in charge of engineering work on the lines in Illinois, Wisconsin, and Minnesota.

When the Bureau of Valuation was organized under the Interstate Commerce Commission in 1913, Mr. Bremner was appointed Assistant District Engineer in the Central District, with headquarters in Chicago. In 1915, he was promoted to District Engineer for this territory, and placed in charge of the administrative detail of field and office work, in connection with the preparation of inventories and reports of all the steam railway properties within

* Memoir prepared by a Joint Committee of the Society and the Western Society of Engineers, consisting of Elmer T. Howson, *Chairman*, Edwin F. Wendt, and W. D. Pence, Members, Am. Soc. C. E., and R. T. Scholes, M. W. S. E.

this District. In the course of his service with the Interstate Commerce Commission, he was also called upon to make a number of special reports on railway properties for the Commission under resolutions of Congress. He terminated this service in December, 1921, when the District Offices were closed and the forces concentrated at Washington, D. C. After engaging for a short time in consulting practice, he returned to the Chicago, Burlington and Quincy Railroad Company, with which he remained until his death.

Mr. Bremner always took a keen interest in the work of Engineering Societies. He joined the Western Society of Engineers on October 8, 1887, and participated in the organization of the American Railway Engineering Association in 1897-99. He was a member of the Special Joint Committee of the Society and the American Railway Engineering Association on Stresses in Railway Track, from its organization, in 1913, until his death. In 1906, the Western Society of Engineers awarded him the Octave Chanute Medal for the preparation and presentation of a paper of special merit on the subject of "Areas of Waterways for Culverts and Bridges." Mr. Bremner took a particularly active part in the work of the American Railway Engineering Association, serving at various times as a member of the Roadway and Track Committees, as well as Chairman of the Roadway Committee for three years. He was elected Treasurer of the Association in 1911, in which capacity he served until his death.

Gifted with strong personal initiative, Mr. Bremner was nevertheless a good soldier in carrying out the wishes of a superior authority, when the decision and plan of action were made. It was his habit when approaching new problems to form his judgment carefully, at times even slowly; with this quality of standing firmly for his beliefs, he did not hesitate to recast his views in the light of new facts, of which he was ever in search on his own account. Of his many strong personal qualities none is better remembered than his sturdy sense of business integrity, a steadfast loyalty to his friends, and, above all, his fine devotion to his family circle.

As a boy, Mr. Bremner united with the Presbyterian Church, which organization received his unfailing loyalty throughout his life.

He was married on September 5, 1888, at Red Oak, Iowa, to Louie A. Stephenson, who, with two sons, Charles W. and George H., and a daughter, Mrs. Clarence Schaeffer, of Chicago, survives him; he is also survived by a sister, Mrs. O. A. Byington, of Iowa City, Iowa, and a brother, William H. Bremner, of Minneapolis, Minn.

Mr. Bremner was elected a Member of the American Society of Civil Engineers on December 6, 1899.

WILLIAM CHARLES BURKE, M. Am. Soc. C. E.*

DIED JULY 21, 1928.

William Charles Burke was born near Montreal, Que., Canada, on February 1, 1857. He was the son of James and Mary (MacDonald) Burke. His father was of Irish ancestry and his mother of Scotch descent.

When Mr. Burke was fifteen years of age his family moved to Kansas City, Mo. where he completed his preliminary education. In 1874 and 1875, he attended Kansas University as a student of Civil Engineering, and, during summer vacations, received his first practical engineering experience, acting as Assistant to the City Engineer of Kansas City.

From 1876 to 1880, he was employed by the Union Pacific Railway Company and rose rapidly from a subordinate position to that of Locating Engineer. He made numerous location surveys for this Company in Western Kansas and Colorado during the period when Indian uprisings were not unusual, and military escorts were necessary.

During 1880 and 1881 he located and built the Missouri, Kansas, and Texas Railway from Fort Worth to Taylor, Tex., and, in 1882, was in charge of the erection of the Red River Bridge for the Cotton Belt Railway at Texarkana, Tex. This work was carried on under serious difficulties, as malarial fever was very prevalent and Mr. Burke was forced, on the completion of the railroad, to seek a more healthful climate. He went to San Marcial, N. Mex., as Division Engineer in charge of maintenance of way, of the Rio Grande Division of the Atchison, Topeka and Santa Fé Railway, in which position he remained until 1884 when he was appointed Assistant Chief Engineer of the Kansas City, Clinton, and Springfield Railway. He located and built this line from Kansas City to Springfield, Mo.

He spent the next two years, 1885 and 1886, with the Union Pacific Railway Company, locating a line from Salina to Lincoln, Kans., and relocating and constructing the line from Denver to Boulder, Colo. On the completion of this work, in 1887, Mr. Burke returned to the Atchison, Topeka and Santa Fé Railway Company, and located and built its main line from Fort Worth to Purcell, Okla. After this work was finished, he located and built the line of the Missouri Pacific Railway from Marquette to Gypsum City, Kans.

From 1888 to 1890, he was associated with two of his brothers-in-law under the firm name of Colt, Reinhart, and Burke, Contractors. They constructed the Missouri Pacific Railway from McCracken, Kans., to Pueblo, Colo., and the Fort Lyon Irrigation Canal in the Arkansas Valley, Colorado. The firm promoted, designed, and built the Rocky Ford, Colorado, Highline Canal, and numerous small projects, and was also engaged in the mercantile business, milling, and sheep raising. During the period from 1883 to 1890 Mr. Burke was also associated with his brother, Thomas Burke, in the cattle business, their ranch headquarters being thirty miles northwest of Prescott, Ariz.

* Memoir prepared by Morris C. Burke, Assoc. M. Am. Soc. C. E.

From 1890 to 1897, Mr. Burke lived at Las Animas, Colo., engaging in various activities. He made a report to the U. S. Department of the Interior as to irrigation possibilities in the Ute Indian Reservation. He was Consulting Engineer for the Great Plains Water Storage Company of Holly, Colo., and served for years as Receiver of the Fort Lyon Canal Company. He also made various reports of railroad and irrigation projects in the West and in Mexico.

In 1898 and 1899, as a member of the contracting firm of J. B. Colt and Sons Company, he built the St. Louis and North Arkansas Railway from Eureka Springs to Harrison, Ark. In 1901, Mr. Burke located the Eastern Oklahoma Branch of the Santa Fé Railway from Newkirk to Pauls Valley, Okla., and revised the location of the main line from Arkansas City, Kans., to Newkirk, Okla. During 1902 and 1903, as Division Engineer, he constructed the Missouri, Kansas, and Texas Railroad from Coffeyville, Kans., to Oklahoma City.

In 1904 Mr. Burke was appointed City Engineer of Oklahoma City, Okla., and remained in this position until 1911. During these years, the city grew very rapidly and the problem of keeping the improvements apace with the growth was tremendous.

From 1911 to the time of his death, he lived and maintained his office in Oklahoma City, and his consulting practice covered numerous projects of all descriptions, the more important of which were, as follows: Chief Engineer, Municipal Engineering and Construction Company; Consulting Engineer, State of Oklahoma, in the Two-Cent Rate Case; Chief Engineer, Cameron County Water Improvement District No. 1, Harlingen, Texas; Chief Engineer, Lee Land Company, Lower Rio Grande Valley, Texas; Chief Engineer, Rancho de Santa Maria, Texas; Chief Engineer, Municipal Excavator Company, Oklahoma City; Chief Engineer, State Conservation Commission of Oklahoma; Chief Engineer, North Canadian River Flood Control District No. 1, Oklahoma City; Consulting Engineer, Oklahoma Natural Gas Company; Consulting Engineer, Atchison, Topeka and Santa Fé Railway; Consulting Engineer, Citizens Committee of Oklahoma City on Union Railway Station controversy; and senior member of the firm of Burke Engineering Company.

Mr. Burke was married in 1887 to Daisy May Colt, of Clinton, Mo. He is survived by his widow, two sons, Morris Colt Burke, Assoc. M. Am. Soc. C. E., of Oklahoma City, and James MacDonald Burke, of La Jolla, Calif., and two daughters, Mary Frances Burke and Harriett Elizabeth Burke.

Mr. Burke's life was a splendid example of devotion to duty, to his profession, to his employers, and to his family. He was admired and loved by all who knew him and numbered his friends by the thousands throughout the Southwest. The history of his life is practically the history of the building of the West, and a personal narrative of his career would be rich with romance and adventure.

Mr. Burke was elected a Member of the American Society of Civil Engineers on April 12, 1926.

EUGENE DICKINSON BURNELL, M. Am. Soc. C. E.*

DIED FEBRUARY 20, 1928.

Eugene Dickinson Burnell was born at Steilacoom, Wash., on December 11, 1884, the son of Dr. A. T. and Mary (Frayer) Burnell.

Mr. Burnell attended Cornell University, at Ithaca, N. Y., from which he was graduated with the degree of Civil Engineer in 1906. From July, 1906, to December, 1908, he was employed on drafting and estimating by Hugh L. Cooper, M. Am. Soc. C. E., Consulting Engineer, of New York, N. Y., and, later, as Foreman on hydro-electric power plant construction, including the design of hydro-electric power plants.

He left the employ of Mr. Cooper in January, 1909, to fill an engagement as Assistant Engineer for the City of Pensacola, Fla.; he did not remain long in this work, however, because of his appointment to a position, in which greater experience could be obtained, with the Isthmian Canal Commission on the construction of the Panama Canal. He went to Panama in February, 1909, remaining there until January, 1910, as Assistant Engineer in the Department of Masonry Design, being particularly concerned with the design of the locks, gates, and valves, at Culebra.

In February, 1910, Mr. Burnell accepted a position with the J. G. White Engineering Company, of New York. At first, his duties included reporting on proposed hydro-electric developments, designs, and estimates. Later, he was appointed Assistant Superintendent of Construction on the 30 000 h.p. hydro-electric plant of the Canadian Light and Power Company, at Montreal, Que., Canada.

On the completion of this work in February, 1913, Mr. Burnell accepted a position as Construction Engineer for the Phœnix Construction Company, at Grace, Idaho, on the construction of the Utah Power and Light Hydro-Electric Development, one of the properties of the Electric Bond and Share Company. On this work he made evident his capabilities as a Cost Analysis Engineer.

Mr. Burnell's work in Idaho was completed in the spring of 1914, and he then accepted a position with the Power Construction Company of Shelburne Falls, Mass., as Construction Superintendent in charge of building ten miles of a standard-gauge railroad. He finished this work in September, 1914. In January, 1915, he was appointed Superintendent of Construction for the Aqueduct Construction Company, Winnipeg, Man., Canada, on the construction of ten miles of a concrete aqueduct, which position he retained until the work was completed in April, 1916.

When Mr. Burnell returned to New York, he was employed by the Agency of the Canadian Car and Foundry Company of New York, as Assistant Works Manager, at its large shell and shrapnel loading plant at Kingsland, N. J. It was during his engagement on this work that he showed his forcefulness as an efficient Production Manager, and it was largely through

* Memoir prepared by Joseph D. Evans, Esq., New York, N. Y.

his technical ability, energy, and painstaking efforts that the Company was able to fulfill the contract on which the production had fallen far behind. The penalties that were being enforced would have meant a great financial loss to the Company. Through Mr. Burnell's efforts, however, work under this contract with the Russian Government for loading 2 500 000 18-lb., high-explosive shells and 2 500 000 rounds of shrapnel, at an aggregate price of $83 000 000, was brought up to large quantity production and the Company's penalties were soon cancelled by very substantial bonuses, all of which were in excess of the original contract price.

In April, 1917, soon after the United States entered the World War, the Evans Engineering Corporation, of New York, which had been formed for the purpose of loading 18-lb., high-explosive shell for the United States Army, employed Mr. Burnell as Chief Engineer in charge of design and construction of the $800 000 shell-loading plant which this Company was building at Old Bridge, N. J. Later, he was placed in charge as Works Manager. Although this plant, which was a privately owned corporation, loading shells on a unit-price basis, with contracts from the United States Ordnance Department, was the smallest of the shell-loading plants in the United States, it holds the record for loading the greatest number of 18-lb. shells (75-mm. high-explosive) in any one day, month, or for the duration of the World War, and this work was carried on under the able direction of Mr. Burnell.

One of the financial interests in the Evans Engineering Corporation caused the Atlantic Loading Company to be formed, for the purpose of constructing an $18 000 000 to $25 000 000 high-explosive shell-loading plant for the United States Government. This plant was constructed later and operated at Amatol (Hammonton), N. J. In conjunction with his duties as Works Manager of the Evans Engineering Corporation plant at Old Bridge, Mr. Burnell was appointed Chief Engineer on design for the Amatol plant of the Atlantic Loading Company. When this latter Company's plant was sufficiently constructed to permit operations of loading high-explosive shell, Mr. Burnell devoted his entire time to serving as Works Manager at Amatol. Of the four plants of this type which the Government constructed, this one was reputed to have been the largest. It had a capacity of turning out more than 150 000 high-explosive shells daily, in sizes ranging from 18 lb. (75 mm.) to 80 lb. At the termination of the war, the Government selected this plant for carrying on several years' operations, scrapping the others which had been constructed under agency form of contracts. The Atlantic Loading Company terminated its work in the summer of 1919. Mr. Burnell's work during the war stands out eminently as an accomplishment second to none for its efficiency and harmonious operation.

In 1919 Mr. Burnell joined the Overman Cushion Tire Company, of New York, as Assistant to M. C. Overman, President; later, he was made a Vice-President. As is so marked in his career, he again showed himself to be of a constructive turn of mind and was instrumental in systematizing the entire operations of this Company.

In August, 1917, Mr. Burnell was married to Augusta A. Evans, who survives him. He is survived also by his father and mother and three sisters,

Elizabeth Burnell, of Los Angeles, Calif., Mrs. Bernice Maricle, of Everson, Wash., and Mrs. Esther Mills, of Longs Peak, Colo.

All those who knew Mr. Burnell soon learned to appreciate that he was an engineer and business man of ability, one whose loyalty and integrity ever could be counted upon. He was quiet, unassuming, and always had a kind word for every one with whom he came in contact.

He was a member of the Cornell University Club, New York, the Rod and Bob Club of New York, the Rockwood Hall Country Club, the Society of Automotive Engineers, and the American Society of Mechanical Engineers.

The passing of Mr. Burnell leaves a vacancy in the hearts of all his friends, and takes from the world an engineer and operating manager who, had he lived, might have accomplished still greater things in useful construction.

Mr. Burnell was elected an Associate Member of the American Society of Civil Engineers on June 4, 1913, and a Member on September 9, 1919.

HENRY JACKSON BURT, M. Am. Soc. C. E.*

DIED JULY 28, 1928.

Henry Jackson Burt, born February 6, 1873, at Urbana, Ill., was the son of H. Jackson Burt and Isabelle (Dunlap) Burt. His father died when he was six months old, which, to a great extent, necessitated his self-education.

Mr. Burt's entire education was received in the schools at Urbana. He was graduated from the University of Illinois in 1896, with the degree of Bachelor of Science, and was awarded the degree of Civil Engineer in 1914. During his school years of 1892 and 1893 he was engaged in surveying and modeling a relief map of the State of Illinois for the Columbian Exposition. During the summer of 1895 he served as Instrumentman and Inspector on sewer construction at Urbana.

After his graduation from the University of Illinois in 1896, he entered the employ of the Gillette-Herzog Manufacturing Company, of Minneapolis, Minn., in its New Orleans Office, designing and estimating steel structures of all kinds. On some of these structures Mr. Burt also acted as Inspector of Material and Superintendent of Erection. In December, 1898, he resigned his position with the Company to accept one with the Chicago and North Western Railway Company as Assistant Engineer on the construction of new lines in Iowa; and in January, 1900, he was placed in charge of surveying for track elevation in Chicago, Ill.

From September, 1900, to June, 1901, Mr. Burt was engaged as Assistant Professor of Civil Engineering at Iowa State College, Ames, Iowa, under Anson Marston, President, Am. Soc. C. E. In July, 1901, he entered the employ of the American Bridge Company as Contracting Manager, serving one year at St. Louis, Mo., three years at Salt Lake City, Utah, and six years at Denver, Colo., resigning in 1911 to accept a position as Chief Structural Engineer with Holabird and Roche (now Holabird and Root), Architects, Chicago. Among the notable large Chicago buildings designed under his supervision while he was employed in this capacity, or as Consulting Engineer, are the Sherman, Palmer, Morrison, and Stevens Hotels, the Tribune Tower, the Roanoke Tower, the Illinois Stadium, at Champaign, Ill., and the Soldiers' Field Stadium, at Chicago.

During the World War, 1917-1919, Mr. Burt was with the Construction Division of the United States Army, at Washington, D. C., in charge of plans for emergency construction, as Major in the Quartermaster Corps. Later, he was made Lieutenant-Colonel in the Officers Reserve Corps. After the war, he returned to practice as a Consultant in civil and structural engineering, specializing in heavy building construction, both foundations and superstructures.

He was the author of "Steel Construction", widely used as a text and reference book. He had an exceptionally well-grounded knowledge of engi-

* Memoir prepared by a Joint Committee of the Society and the Western Society of Engineers, consisting of Albert F. Reichmann, *Chairman*, August Ziesing, and F. E. Brown, Members, Am. Soc. C. E., and A. N. Talbot, Past-President, Am. Soc. C. E.

neering principles, with a thorough understanding of their practical application, and was a capable organizer and executive. With these attainments he combined good business reasoning and fair play which accounted for his cordial relations with all with whom he came in contact. His modest and unassuming manner of guidance, which was never dictatorial, endeared him to his many friends and associates. He was modern and progressive, but never radical nor over-confident, and with his keen analytical mind he would attack a problem in a manner which seemed to sense a solution by instinct.

Mr. Burt was a loyal and alert citizen, always interested in public questions, and gave much of his time, whenever he could be of service, to the community in which he lived. His work as Chairman of the Legislative Committee of the Western Society of Engineers during 1914 and 1915 is especially commendable, because it resulted in the Engineers' License Law of the State of Illinois. To the time of his last illness, he was a member of the Wheaton Sanitary District Board. He also served as Chairman of the Board of Trustees of the First Presbyterian Church. His high ideals and genial personality were an inspiration, not only to his friends, but especially to his family; and his beautiful home life reflected these influences.

In 1900, Mr. Burt was married to Edith Fleming, at Champaign, Ill. He is survived by his widow and five children, Henry Jackson, Luella, Jesse Fleming, Morris Nathan, and Helen Edith.

He was a member of the Western Society of Engineers (Past-President, 1917), American Society for Testing Materials, American Society for Heating and Ventilation, American Concrete Institute, and the Society for the Promotion of Engineering Education. He was also a member of the University Club and the Illini Club of Chicago, the Chicago Engineers Club, and the Masonic Fraternity.

Mr. Burt was elected a Member of the American Society of Civil Engineers on March 1, 1905.

DAVID SYLVANUS CARLL, M. Am. Soc. C. E.*

DIED NOVEMBER 5, 1928.

David Sylvanus Carll was born in Huntington, N. Y., on March 21, 1855, the son of David and Mariette (Parsons) Carll. He was graduated from the Huntington Union School in 1875.

Mr. Carll's professional work began immediately after his graduation, his early experience consisting of surveying in the vicinity of his home. From 1877 to 1881, he was Surveyor to various commissioners of highways. Subsequently, he was Transitman and Assistant Engineer for the Long Island Railroad Company, Assistant Engineer for the New York, West Shore, and Buffalo Railway Company, and he was also engaged in practice as a Civil Engineer and Surveyor in Saugerties, N. Y.

From 1884 to 1888, Mr. Carll was with the Department of Public Works of New York City, in connection with the construction of the new aqueduct and reservoirs. In March, 1887, he went to Kansas City, Mo., where he was engaged on the construction of the cable railway lines, his first employment being with the Grand Avenue Railway Company, and, later, as Assistant Engineer with Knight and Bontecou, Consulting Engineers. From October, 1889, to 1890, he was Superintendent of Grading and Construction in the City Engineer's Office in Kansas City.

In November, 1890, Mr. Carll began his career as Engineer Executive for the Washington and Georgetown Railroad Company, now The Capital Traction Company, of Washington, D. C. His first work in Washington was in charge of installing the cable system on the lines of that Company. On the completion of the work in 1892, he was made Chief Engineer and continued in that position, in charge of engineering work, until 1908. Subsequent to 1895, he became the Superintendent of the Company, retaining his position as Chief Engineer, and was responsible for all the Company's Operating as well as its Engineering Departments. During the construction of the cable road his duties included not only the construction of the cable lines themselves, but of a cable power station and office buildings on Pennsylvania Avenue, on the site of the present District Building. He was also engaged on the construction of several car barns and other structures.

After the destruction of the Cable Power Station by fire in September, 1897, the lines of the Company were electrically equipped; this was the first instance of the conversion of cable lines to the underground conduit system. Mr. Carll was in charge of all this work as Chief Engineer, including, in addition to the reconstruction of track, the building and equipment of a direct-current power station, the rebuilding of numerous car barns, and the design and purchase of rolling equipment. This work was done principally during 1897 and 1898, and at this time Mr. Carll's responsibilities also included the operation of all the Company's street railway lines, which for a time included operation by horse cars, cable, over-head trolley, and underground conduit. In

* Memoir prepared by J. H. Hanna, M. Am. Soc. C. E.

February, 1908, Mr. Carll was made a Director of The Capital Traction Company and elected to the office of Vice-President and General Manager. He remained a Director and Vice-President until his death, having given up his more active duties with the Company in 1916.

During Mr. Carll's long residence in Washington he showed great interest in the development of the Capital City by active association with many civic organizations. He had been a prominent member of the Board of Trade for many years and was Vice-Chairman of its Committee on Law and Order. He was a Charter Member, and served on the first Board of Directors, of the Washington Society of Engineers; he also served as Vice-President in 1908 and as President in 1909. He had been prominently active in the District of Columbia Section of the American Society of Civil Engineers since its organization and served as Chairman in 1920. He was a member of the Cosmos Club, of Washington, and many other civic and welfare organizations.

The Mt. Pleasant Congregational Church of Washington was, next to Mr. Carll's business and profession, his greatest interest. A member of the church since 1894, he was a Trustee for more than twenty years and Chairman of its Building Committee during the erection of its present structure.

Mr. Carll was married on February 22, 1883, to Maria B. Dawes, who died in 1922, leaving one daughter, Mrs. Ernest D. Roberts, of Washington. In 1925, he was married to Eliza R. Davis, of Auburn, N. Y.

To those who were fortunate enough to be associated with David S. Carll and to have his friendship, his outstanding characteristic was his humanity and love for his fellow man. An able engineer, a forceful, energetic executive, always carrying heavy responsibilities, he never failed to find time for a word of encouragement or more substantial help for any who came to him. The love of the thousands of men and women who had the privilege of working under his direction is the outstanding feature of his successful career—his bequest to them, the memory of a Christian gentleman.

Mr. Carll was elected a Junior of the American Society of Civil Engineers on March 7, 1888; an Associate Member on July 1, 1891; and a Member on October 7, 1896.

JOHN CHARLES CHURCHILL, M. Am. Soc. C. E.*

DIED APRIL 19, 1928.

John Charles Churchill was born at Oswego, N. Y., on September 20, 1856, the second son of Judge John C. and Kathleen Sprague Churchill. His father was one of Oswego's leading citizens, having been a Representative in Congress and also a Justice of the Supreme Court. He received his preparatory education in the public schools of Oswego, and was graduated, in 1878, from Lafayette College, Easton, Pa., with the degree of Mining Engineer.

From March, 1879, to July, 1881, Mr. Churchill was Surveyor and Draftsman for the Federal Government on harbor improvement work on Lake Erie. During the remainder of 1881, and until the end of 1884, he was Assistant Engineer for the Oregon Railway and Navigation Company, for two years in charge of the construction of bridges and trestles, and of tracklaying on the Baker City Branch. He returned to New York State in 1885, and from May to October was in charge of harbor improvement work at Ogdensburg, N. Y. From October, 1885, to October 1886, he was Assistant Engineer on the Rome, Watertown and Ogdensburg Railroad, in charge of the construction of a ferry slip at Ogdensburg, and of construction work on the branch from Norwood to Massena, N. Y. From October, 1886, to August, 1887, he was Assistant Engineer for the Federal Government on various surveys for harbor improvements in Northern New York and Vermont. In August, 1887, he resigned from the Government service to enter the contracting business, continuing in this work until July, 1889.

From July, 1889, to January, 1891, Mr. Churchill was Engineer and Assistant to the Building Superintendent on the construction of the St. Lawrence State Hospital, at Ogdensburg. In 1892, he acted as City Engineer of Ogdensburg, during which service he designed and constructed the South Street main sewer, and rebuilt the sea wall in the Oswegatchie River.

In 1892, he returned to the Federal Government Service in charge of harbor improvement work at Ogdensburg, continuing until August, 1895, when he was made Assistant Engineer in charge of the St. Lawrence River improvement. From January, 1896, to July, 1898, he was in charge of examination and surveys on Lake Champlain, the St. Lawrence River, and Lake Ontario, having been also from January to May, 1898, in charge of the U. S. Engineer Office at Oswego, N. Y., during the absence of the District Engineer. In July, 1898, the Oswego District was combined with the Buffalo, N. Y., District, and Mr. Churchill was given charge of the Oswego Branch Office, in which position he continued until it was again separated from the Buffalo District in August, 1909. When the Oswego District Office was re-established in August, 1909, he became Principal Assistant to the District Engineer, the District including all harbors on the south

* Memoir prepared by Lynn L. Davis, M. Am. Soc. C. E.

shore of Lake Ontario and the St. Lawrence River. In May, 1913, the Oswego District was again combined with the Buffalo District, and Mr. Churchill for the second time was made Assistant Engineer in charge of the Oswego Sub-Office, in which capacity he continued to serve until his death.

In addition to this regular service he was employed in a consulting capacity as follows: By the State of New York as Consulting Engineer and Expert Witness in matters pertaining to water power on the Oswego River; and for a time as Chairman of the Water Commission of the City of Oswego. He also served as Fuel Administrator of the City of Oswego during the World War. He had applied for a commission in the Corps of Engineers during the war, but his request was refused on account of his age. All the members of his family were engaged in some capacity in war work, his three sons having been in active service.

Mr. Churchill served the Federal Government on harbor improvement work for more than thirty-eight years, on Lakes Erie, Ontario, and Champlain, and on the St. Lawrence River. He designed and built many of the breakwaters and piers on Lake Ontario and the St. Lawrence River. His knowledge regarding the needs of commerce on these lakes and of the character and condition of all the structures in the former Oswego U. S. Engineer District was very valuable, and his advice was sought on all projects for improvement or maintenance.

He died at his home in Oswego, on April 19, 1928, and is survived by his widow, formerly Mary Beekman Westbrook, three sons, John W. Churchill, of Washington, D. C., Maj. Lawrence S. Churchill, Air Corps, U. S. A., Langley Field, Va., and Walter A. Churchill, of Salem, Mass.; one brother, Lawrence W. Churchill, of White Plains, N. Y., and St. Petersburg, Fla.; and one sister, Mrs. R. R. Fisher, of Oswego.

Mr. Churchill was one of God's noblemen. His kind and genial disposition and splendid upright character won for him the highest respect of all who were privileged to know him. In his business and professional relations he was held in the highest regard by all who were associated with him. His City will miss his wise and judicial counsel and the Federal Government will feel the loss of a valued and able servant whose wisdom was always sought in all matters of policy and details of Federal work in his District.

Mr. Churchill was elected a Member of the American Society of Civil Engineers on February 1, 1899.

WALTER WOODBRIDGE CLIFFORD, M. Am. Soc. C. E.*

DIED JANUARY 19, 1927.

Walter Woodbridge Clifford was born on February 15, 1888, at Wiscasset, Me. His parents died when he was a small boy, and he went to Hyde Park, Mass., to live with his uncle and aunt, who raised him. He attended the public schools of that city and was graduated from the Hyde Park High School in 1905. He then entered the Massachusetts Institute of Technology, receiving his degree of Bachelor of Science in the Department of Civil Engineering in 1909. While in attendance at the Institute he was a member of the Civil Engineering Society, and took an active interest in student military affairs.

In the summer of 1909, following his graduation, Mr. Clifford was in the service of the old Harbor and Land Commission of Massachusetts, as Surveyor in connection with the proposed dredging of harbors along the Massachusetts Coast. In the fall of that year he returned to the Massachusetts Institute of Technology as an Assistant in the Department of Civil Engineering, where he remained for two years. During the summer of 1910 he was employed as Resident Engineer by the Aberthaw Construction Company, which was building a concrete stand-pipe for the Town of Westerly, R. I.

After two years of teaching, he entered the employ of the United States Forestry Service as a Hydraulic Engineer, but he remained in the Service only a few months. From May to September, 1912, he was employed by Metcalf and Eddy as Resident Engineer in charge of the water supply and sewage disposal of the new Technology summer surveying camp, which was then being built at East Machias, Me. On the completion of this work he was employed for a few months by the U. S. Navy Department, at the Charlestown Navy Yard, as a Designing Engineer.

In 1913, Mr. Clifford entered the Engineering Department of Stone and Webster, Incorporated, and was assigned to the staff selected for the engineering design of the new Technology buildings at Cambridge. He did some valuable work in the solution of many difficult problems which arose in connection with the foundations and superstructure of these buildings. In 1915 he was employed as a Structural Engineer by Densmore and LeClear, and remained with this architectural firm for about two years. From 1917 to 1921, he was with the Harry M. Hope Engineering Company, first, as Chief Draftsman, and, later, as Designing Engineer on many important structures, among which was the large plant of the United Drug Company, at St. Louis, Mo.

From 1921 until his death, which occurred suddenly from heart failure at his office in Boston, Mass., he had been associated with Mr. S. G. Roeblad, under the firm name of Clifford and Roeblad, Consulting Engineers. The principal work of this organization consisted in designing industrial plants and garages, and acting in the consulting capacity of structural engineers for architectural firms. Mr. Clifford was engaged personally as an expert in Court proceedings, and also as arbitrator in connection with the interpretation of building contracts.

* Memoir prepared by Charles R. Berry, Assoc. M. Am. Soc. C. E.

He was a member of the Boston Society of Civil Engineers and the American Concrete Institute. He was the recipient from the Society of the Collingwood Prize for Juniors, for his paper, "A Reinforced Concrete Standpipe."* He took an active part in the Boston Society of Civil Engineers, having served on many of its committees. He was Chairman of the Designers Section in 1923 and 1924. At the time of his death he was Chairman of the Welfare Committee and was a candidate for Director of the Society.

Mr. Clifford was the author of many articles on engineering subjects appearing in periodicals, and was one of the Associate Editors of Hool and Johnson's Handbook of Building Construction, and of Hool and Kinne's Structural Engineers' Handbook Library. With C. Hale Sutherland, M. Am. Soc. C. E., he had recently published a textbook entitled, "Reinforced Concrete Design," which was very favorably received.

Not only was Mr. Clifford active in the affairs of his profession, but he also gave generously of his time to Hyde Park Lodge, A. F. and A. M., and Christ (Protestant Episcopal) Church, of Hyde Park, of which he was a member. At the time of his death he held the office of Junior Warden in each of these bodies. He was also a member of the Aleppo Temple Shrine of Boston. He was a Director in the Hyde Park Young Men's Christian Association.

In 1912, he was married to Helen Edelmer Mooar, by whom he is survived. He also leaves two children, James and Deborah, as well as an only brother living in California.

Although of an unassuming disposition, Mr. Clifford took an active part in the various organizations with which he was connected, and was sought out by them for office on account of his fine executive ability. A man of accomplishments, and a true Christian gentleman, Walter Clifford will be missed by a host of friends.

Mr. Clifford was elected a Junior of the American Society of Civil Engineers on November 8, 1909; an Associate Member on January 15, 1917; and a Member on March 16, 1925.

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* *Transactions*, Am. Soc. C. E., Vol. LXXIV (December, 1911), p. 375.

WILLIAM GODLEY COUGHLIN, M. Am. Soc. C. E.*

DIED OCTOBER 12, 1927.

William Godley Coughlin, the son of Samuel Britton and Anna Margaret (Weber) Coughlin, was born on February 15, 1862, in Florence, N. J. His father was of English and Irish ancestry, whereas his mother's family came to the United States from Germany in 1833. The boy received his education in the public and private schools of Florence and, later, attended the Farnum School, at Beverly, N. J., preparatory to entering Lehigh University, at Bethlehem, Pa.

During this period, his father decided to join the Collins Expedition of 1878, with the purpose of engaging in railroad construction in Brazil. As the prospect of such an adventurous trip to South America appealed strongly to the lad's imagination, he went with the Expedition and obtained the position of Rodman on the Madeira and Mamore Railway around the Falls of the Upper Madeira in Brazil. The Expedition seems to have been ill-fated, for those of the party sailing on the second ship, the *Metropolis*, were lost at sea, Mr. Coughlin having escaped that fate by his persistent petitions to be allowed to accompany his father.

On his return to the United States, after a year and a half of service in South America, Mr. Coughlin became engaged in preliminary surveys for a railway from Pittston to Hawley, Pa. He then entered the service of the Pennsylvania Railroad Company, which was the beginning of a life service in the Engineering, Maintenance of Way and Structures, and Transportation Departments of that System. He was employed, successively, as Rodman, Levelman, and Transitman on the following work: Preliminary surveys and location for a branch line from Baltimore, to Catonsville, Md.; survey for a proposed pleasure railway through Fairmount Park; re-location and grade revision of the Western Pennsylvania Railroad; preliminary surveys and location of the Cornwall and Lebanon Railroad; revision of alignment on the main line of the Pennsylvania Railroad; location and construction of the Philadelphia and Long Branch Railway from Whitings to Bay Head, N. J.; fourth track work on the Western Division of the Pennsylvania Railroad; and location and construction of a connecting line between Dauphin and Rockville, Pa.

In March, 1883, Mr. Coughlin entered the Maintenance of Way Department of the Northern Central Railway (Pennsylvania Railroad) as Assistant Supervisor in the office of the Engineer of the Baltimore Division and, in May, 1886, he was appointed Supervisor of the West Jersey and Seashore Railroad (Pennsylvania Railroad) in charge of maintenance-of-way work, which position he held until March, 1893.

The following include some of the positions which Mr. Coughlin held during the period extending from March, 1893, to April 1, 1907: Assistant Engineer on the Baltimore Division of the Northern Central Railway, in full

* Memoir prepared by W. C. Cushing, M. Am. Soc. C. E.

charge of engineering work; Principal Assistant Engineer on maintenance-of-way work on the Erie Division of the Pennsylvania Railroad, including that territory extending from Erie, Pa., to Baltimore, Md., and numerous lateral branches; and Superintendent in charge of operations of the Elmira Division of the Northern Central Railway, with headquarters at Elmira, N. Y.

In April, 1907, Mr. Coughlin was appointed Superintendent of the Erie Division and, in March, 1909, of the Renovo Division, of the Pennsylvania Railroad Company, in which positions he had charge of operations, including engineering work, maintenance of way, bridges, buildings, construction of piers, and dredging.

From November, 1913, until his retirement in October, 1927, Mr. Coughlin held the positions of Engineer and Chief Engineer of Maintenance of Way for the Pennsylvania Railroad Company, having his office at Philadelphia, Pa. Just prior to his retirement, he was occupying the position of Special Agent with this Company.

Mr. Coughlin's faithful and loyal performance of duty appeared everywhere in his work. Gifted with an especially analytical mind, he was constantly active in inventions for the improvement of railroad structures and appliances, with a view to economy and improved elements in the management of his territorial range. He was granted several patents by the United States Patent Office, the last being received a few months before his death. These patents were closely allied to railroad construction and demonstrated the possible achievement of an active imagination coupled with a skillfully managed pencil. His work along this line made evident his earnest efforts to improve the track structure under his official jurisdiction and his vigorous activity in these conceptions during his life time.

Mr. Coughlin was of a modest and retiring disposition, possessing withal a very pleasing personality. Many anecdotes are told by his associates to illustrate his ready wit and humorous outlook. His was a bright, cheery, and sympathetic nature.

He was a member of the Engineers Club of Philadelphia and of the Madeira and Mamore Association, and a communicant of the Protestant Episcopal Church.

As Mr. Coughlin was not married, he lived during the last fourteen years of his life, with his family, at the "old home" in Burlington, N. J.

Mr. Coughlin was elected a Member of the American Society of Civil Engineers on March 4, 1913.

CLARENCE AUSTIN CRANE, M. Am. Soc. C. E.*

DIED DECEMBER 5, 1927.

Clarence Austin Crane was born in New York, N. Y., on October 18, 1874. He was the son of Warren Cady and Caroline (Cleveland) Crane, and was descended from a long line of English ancestors, both great-great-grandfathers having served in the War of the Revolution.

After his preliminary education had been completed in the preparatory schools of New York, Mr. Crane attended the School of Mines, Columbia University, during 1891 and 1892. From October, 1893, to April, 1894, he was a student at Lehigh University, South Bethlehem, Pa., taking special courses in Mechanics.

Previous to this time, Mr. Crane had been connected with the Engineering Department of the New York, Susquehanna, and Western Railroad Company, but after leaving college he resigned this position to become Assistant Engineer with the Department of Parks of the City of New York. In this capacity, he was engaged in many important plans and improvements which included the Harlem River Driveway, the Zoological Gardens of Bronx Park, and the Spuyten Duyvil Parkway.

From 1900 to 1903, he served as Assistant Engineer with the New York Aqueduct Commission, on the construction of Jerome Park Reservoir. In 1903, he resigned from the city service and entered private practice as a Consulting Engineer. As such, he was engaged in surveying in the Boroughs of Manhattan and The Bronx, and elsewhere, and had charge of numerous grading, paving, and sewer contracts, etc. He also served as a City Surveyor in the Borough of Manhattan and was engaged in street grading, etc., on the upper part of New York City.

In 1909, Mr. Crane founded the General Contractors Association, which through his efforts developed into the present influential organization. As Secretary of this Association, he was active in encouraging the enactment of statutes infinitely beneficial to contracting interests as well as to the State in general.

Although unfailing in his devotion to the Association, Mr. Crane's activities were by no means limited to this one undertaking. He was a student of and possessed an intimate knowledge of public affairs. He was a civil engineer with practical experience in engineering problems of design and construction. His advice and counsel were sought by his friends generally, as well as by public officials and members of the Association, to all of whom he was equally helpful in solving difficult civic, economic, and engineering problems.

He had a magnetism and charm of personality that won for him a loyal affection and respect, and made people seek to be with him. He was a devoted husband and father, constantly interested and eager for the welfare and

* Memoir prepared from information on file at Society Headquarters.

happiness of his family. He is survived by his widow, Edna Montgomery Crane, two daughters, Caroline Cleveland and Edna, and one son, Warren Cady, 2d; as well as his father, Mr. Warren Cady Crane, two sisters, Miss Sally B. Crane and Mrs. J. Harvey Birch, and one brother, Mr. Frank W. Crane.

Mr. Crane was a member of Mayor Walker's City Plan Committee, the Horatio Seymour Democratic Club, and the Sheriff's Jury, Third Panel. He was also a member of the Engineers Club, the Sons of the Revolution, St. John's Lodge No. 1, A. F. and A. M., and the Chapel of the Intercession of Trinity Protestant Episcopal Church.

Mr. Crane was elected an Associate Member of the American Society of Civil Engineers on March 5, 1902, and a Member on May 4, 1909. He was also a member of the New York Section.

ELMORE DAVID CUMMINGS, M. Am. Soc. C. E.*

DIED NOVEMBER 17, 1927.

Elmore David Cummings was born on November 14, 1861, in Indiana, Pa. His parents, Joseph and Margaret Shields Cummings, were farmers, and the early life of their only son was spent entirely on the farm. A natural aptitude for study and ambitions along other than farming pursuits brought about on the part of the youth an early determination for a college education along engineering lines. Aided and assisted in his studies by his parents, Mr. Cummings accomplished practically his entire preparation for college at home, save for a part of a year, at the age of 20, which he spent at Washington and Jefferson College.

Mr. Cummings entered Cornell University at Ithaca, N. Y., in 1885 and was graduated as a Civil Engineer four years later with honorable mention for his graduating thesis. He immediately entered the employ of G. W. G. Ferris and Company, of Pittsburgh, Pa., as Inspector of Materials at mills, shop construction of railway bridges, and structural work. The early and thorough insight thus gained in structural steel shop processes, together with the fact that he kept abreast of improvements in shop machinery and methods of fabrication, was of inestimable worth in his life work, which was, for the most part, that of a designer in steel. As he designed, he visualized with rare insight the various shop processes necessary to produce in tangible form the creation of his mind, and the value of his work was greatly enhanced thereby.

From 1891 to 1893, Mr. Cummings was Draftsman and Assistant Designer for the Columbus, Ohio, Bridge Company, and for the late E. L. Corthell, Past-President, Am. Soc. C. E., Consulting Engineer, on large railway bridges. Following these engagements he was an Instructor in Civil Engineering at Washington University, St. Louis, Mo.; an Assistant Engineer to the late J. B. Johnson, M. Am. Soc. C. E., in the laboratory work of the United States Timber Physics Investigations at this same Institution, and after the completion of these investigations, he was again engaged in instructional work, this time as Assistant Professor of Civil Engineering at the University of Maine, Orono, Me. His natural predilection for designing, however, led to his return to that field of endeavor, and from 1896 to 1899 he was Office Engineer and Assistant Designer for the Pittsburgh Bridge Company in charge of contractual relations and design pertaining largely to intricate roof construction and the design of mill buildings.

In 1899, the United States Engineer Office at St. Paul, Minn., engaged Mr. Cummings as Designer and Principal Draftsman under the general supervision of the Engineer Officer for special work of design in connection with the reconstruction of the reservoir dams at the head-waters of the Mississippi River, and the construction of locks and dams between Minneapolis, Minn., and St. Paul. This work demanded many detailed investigations of governing considerations, and was characterized by the most painstaking thoroughness and the exercise of a high degree of originality in design. He was pos-

* Memoir prepared by W. C. Weeks, M. Am. Soc. C. E.

sessed of an unusual ability to convince others that the devices and structures he proposed would function properly when they were fabricated and installed. The signal success of Mr. Cummings' work at this time was an accomplishment which his surviving associates of that period still remember.

From 1901 to 1903, Mr. Cummings was Designing Engineer in the Bridge and Building Department of the Chicago, Milwaukee and St. Paul Railway Company, where his work ranged from the design of culverts, shops, and chimneys, to that of bridges spanning the Mississippi River. In November, 1903, he became Junior Engineer and, later, U. S. Assistant Engineer under the immediate direction of the Chief of Engineers, at Washington, D. C.

Mr. Cummings' work in the office of the Chief of Engineers was particularly concerned with the design, construction, installation, and testing of ammunition hoists and disappearing searchlight towers for sea-coast fortifications. His work demanded special qualifications and an exercise of originality far beyond the capabilities of most designers. His accomplishments in this difficult work were of outstanding merit. He not only made many original studies of highly technical character, but he also developed a considerable number of machines and mechanical structures, and incidentally made numerous inventions on which the Government took out patents in his name. It is noteworthy that without exception the creations of his genius have proved to be eminently successful for the purpose for which they were designed. Save for one instance, descriptions of these devices, although well meriting publication in the technical press, have never appeared, due to their confidential character. By special permission of the Chief of Engineers, a descriptive paper on disappearing searchlight towers prepared by Mr. Cummings was published in February, 1924, by one of the leading engineering weeklies.

During the World War, Mr. Cummings handled the technical features pertaining to fabricated materials valued at many millions of dollars. This material was sent to France and consisted for the most part of framed structural steel for large warehouses designed for ready erection in the field.

Mr. Cummings' active life was devoted to engineering, largely in the field of engineering design in which he particularly excelled. All his work was characterized by thoroughness, simplicity, and adaptability to the purpose for which it was intended. His creations were the result of the most careful and painstaking effort, and they were notable for pronounced simplicity and practicability.

Mr. Cummings, by nature a deeply religious man, was a life long member of the Presbyterian Church. He was married on August 7, 1895, to Jessie Williams, of Cleveland, Ohio, who, with a son, Carl Elmore Cummings, survives him.

He was a member of the Washington Society of Engineers, the Society of American Military Engineers, and the Cornell Clubs of New York, N. Y., and Washington, D. C.

Mr. Cummings was elected an Associate Member of the American Society of Civil Engineers on November 4, 1903, and a Member on June 30, 1911.

WILLIAM SANBORN DAWLEY, M. Am. Soc. C. E.*

DIED MAY 18, 1927.

William Sanborn Dawley, son of John T. and Lemoille (Sanborn) Dawley, was born November 27, 1856, at Stockton, Wis.

He was graduated in 1879 from the University of Minnesota with the degree of Bachelor of Civil Engineering and immediately joined the ranks of those pioneers who were so busily engaged at that time in locating and building railroads.

Mr. Dawley's earlier apprenticeship, like that of most engineers of his time, consisted of a rapid succession of moves from one scene of activity to another. During the ten years immediately following his graduation, he was successively Rodman, Levelman, Transitman, Locating Engineer, Resident Engineer, and Division Engineer on various railroad location and construction projects for the Minneapolis and St. Louis Railroad, the Great Northern Railway, the Minneapolis, St. Paul and Sault Ste. Marie Railway, the Chicago and Indiana Coal Railway, and the St. Paul and Kansas City Railroad Companies. His activities during this period covered the greater part of Minnesota, Wisconsin, Iowa, North Dakota, South Dakota, and Indiana, but there were in addition three outstanding accomplishments.

The location of the Minneapolis and St. Louis Railroad, in 1884, which extended as far west as Watertown, S. Dak., was Mr. Dawley's first work in charge of location. This line ran through the Dakota Hills and necessitated some heavy grading and bridge work. He also located and platted fourteen town sites along this line.

In 1885 and 1886, Mr. Dawley first located and then constructed 65 miles of the Chicago and Indiana Coal Railway, which later became the Indiana Division of the Chicago and Eastern Illinois Railway. The following year he was in charge of construction of 30 miles of line southwesterly from Des Moines, Iowa; this line later was incorporated as one of the component parts of the Chicago Great Western Railroad.

In 1888, Mr. Dawley became Special Assistant Engineer to the Vice-President of the Chicago and Eastern Illinois Railway Company, with which road he remained until 1906, having been made Principal Assistant Engineer in 1890 and Chief Engineer and Engineer of Maintenance of Way in 1894. In many respects, this was the most active period of his life and the one of greatest responsibilities. Railroad mileage was being extended at a rapid pace, and Mr. Dawley had by this time a reputation as a railroad builder which made his counsel much sought. In addition to being in full charge of all engineering work, both construction and maintenance, for the Chicago and Eastern Illinois Railway Company, he located and, subsequently, built the extension from Tuscola to Shelbyville, Ill. Later, he extended the line in Illinois to Thebes on the Mississippi River and to Joppa on the Ohio River. During this 18-year period, he located and constructed more than 500 miles of railroad. In addi-

* Memoir prepared by Baxter L. Brown, M. Am. Soc. C. E., and Milton M. Kinsey, Assoc. M. Am. Soc. C. E.

tion, Mr. Dawley re-designed and rebuilt the yard at Danville, Ill., and the Terminal Yard at Dolton, Ill.

In 1906, Mr. Dawley left the Chicago and Eastern Illinois Railway Company to become Chief Engineer of the Missouri and North Arkansas Railway Company. In the following three years he located 210 miles of new line in Missouri and Arkansas, about one-half of which was in the Ozark Mountains, and required heavy grading and a large number of steel bridges, including a 300-ft. draw-span over the White River. He purchased the right of way for this line and completed its construction, having expended in three years more than $5 000 000.

In 1909, at the earnest solicitation of Onward Bates, Past-President and Hon. M. Am. Soc. C. E., Mr. Dawley signed a contract as Chief Engineer of the newly formed Yunnan, Szechuan and Teng Yueh Railway Company of China. Mr. Bates had just returned from a trip to China and had been asked to assist in the selection of an engineer for the railway. On his arrival at Yunnan Fu in Southwest China, Mr. Dawley found it necessary to train twenty young Chinese in the art of surveying before he was able to commence the work of location. His success at this unusual engineering task is evidenced by the fact that in the next four years he had completed the exploration and reconnaissance surveys for more than 1 400 miles of railway and the preliminary and final location surveys and estimates for 600 miles of additional track. All this work was in the extremely mountainous district of Southwestern China.

The unsettled condition of the country and the Revolution which resulted in the formation of the Chinese Republic produced long periods of inactivity, however, and the railroad was never constructed. After a number of rather hazardous experiences, Mr. Dawley returned home in 1913.

On his return, he entered private practice as Consulting Engineer and assisted Bion J. Arnold, M. Am. Soc. C. E., in a study and report on the railroad terminals of Chicago. He made similar studies with Mr. Arnold at Flint, Mich., in 1915, and at Baltimore, Md., in 1917. In 1918, he became associated with C. E. Smith and Company, Consulting Engineers, of St. Louis, Mo., and was in charge of many problems of river freight-handling facilities as well as railroad, industrial, and civic problems.

Although more than 60 years old at the time of the World War, Mr. Dawley made application to enlist in the Army and was commissioned a Major only to have the commission rescinded immediately due to his age. He was employed by the Government, however, as Consulting Expert on the Army Base at New Orleans, La., and, later, was assigned to the United States Railroad Administration to assist in settling claims arising from the Government operation of the railroads.

Following the war, he became greatly interested in the Federal valuation of railroads. His personal connection with the original construction of many miles of railroad, coupled with his intimate and well retained knowledge of the cost, made him much in demand to present expert opinion before the Interstate Commerce Commission. It was while on an inspection trip, preparing for a

railroad valuation case, that Mr. Dawley suddenly became ill on the train, and died shortly thereafter.

Mr. Dawley was particularly fond of young engineers, and many of them have been made better engineers and better men by the kindly and unassuming way he had of explaining to them the problems that he had faced, as well as by the way in which he always could make clear the ethics of a situation.

That equals and superiors as well as subordinates loved him and trusted him cannot better be told than in that which has been written already by a Railroad President with whom he was associated for years:

"With his ability there is coupled a kindliness of heart and temperament that makes him particularly useful in meeting strangers and adapting himself to changing conditions, and he has ability without a display of vanity and will never offend even the poorest laborer, nor be a party to corruption no matter from what source it may arise."

Mr. Dawley was a member of the Western Society of Engineers, the St. Louis Engineers' Club, the St. Louis Railway Club, the Circle Club, and the City Club of St. Louis. He was active in the founding, and was a Charter Member, of the American Railway Engineering Association and served as the first Treasurer of that Association, holding the position from 1899 to 1909.

In 1890, Mr. Dawley was married to Angie Louise Havens. Mrs. Dawley and a daughter, Helen Louise, survive him.

Mr. Dawley was elected a Member of the American Society of Civil Engineers on October 4, 1905.

JAMES HOPKINS DOUSMAN, M. Am. Soc. C. E.*

DIED JANUARY 5, 1925.

James Hopkins Dousman was born at Milwaukee, Wis., on September 24, 1863. After completing the courses in the public schools at Milwaukee, he entered the University of Wisconsin, at Madison, Wis., from which he was graduated with the Class of 1884. Even in early life he displayed an energy which was always one of his most prominent attributes. By utilizing all his vacations and other spare time, he managed to acquire the equivalent of one and one-half years of actual experience. During this period he worked for the County Engineering Department at Milwaukee, the Chicago, Milwaukee, and St. Paul Railroad Company, and for the late C. B. Davis, M. Am. Soc. C. E., a Consulting Engineer, at Chicago, Ill.

After graduation, his first position was that of Transitman with the United States Engineer Corps; subsequently, for three years he acted as Deputy County Surveyor of Edwards County, Texas; and then from 1888 to 1892 as Engineer for the Land and Station Department of the Galveston, Harrisburg, and San Antonio Railroad Company.

In 1893, Mr. Dousman organized the firm of Dousman and Sheldon, Engineers, of Milwaukee, which firm was succeeded in 1895 by that of Dousman and Austin, Consulting Engineers, also of Milwaukee. During this period of private practice, Mr. Dousman was connected with a number of important projects, including work for the Chicago, Milwaukee and St. Paul Railroad Company and water-works and sewage design for the Cities of South Milwaukee and Delevan, Wis., and others. Also, during this period he invented an automatic pressure regulating valve for stand-pipes and other hydraulic work. These valves were used in a number of cities, and a special installation was made for the Cataract Construction Company, at Niagara Falls, N. Y. Mr. Dousman also invented the "Dousman hangers" for carrying falsework on steel construction when concrete or terra cotta is used for fireproofing. In 1898, he became associated with the Milwaukee Engineering Company and held the office of Secretary of that Company until 1902. From 1902 until 1906, he was again engaged in private practice in Wisconsin, specializing in water-works and reinforced concrete design.

In 1906, he took charge as City Engineer of an important water-works improvement at Trinidad, Colo., which work he finished in a highly creditable manner. He then went to Kansas City, Mo., as Representative of the American Well Works of Aurora, Ill., in Kansas, Oklahoma, Western Missouri, and Southern Nebraska, and continued this connection until his death.

There were few, if any, who could equal Mr. Dousman in the energetic way in which he not only sought and obtained contracts, but also in the way these contracts were completed. In addition to serving his firm well, he rendered a great service to the purchaser and the public in expert and trustworthy counsel and advice. He was successful, and universally liked and

* Memoir prepared by N. T. Veatch, Jr., and Bert C. Wells, Members, Am. Soc. C. E.

respected, and his passing is deeply mourned by all who knew him. He not only had the respect and friendship of those to whom he was selling, but also that of his competitors. Until his last sickness, Mr. Dousman's entire life as an engineer had been spent in an active and earnest effort to discharge each task undertaken with credit, and in this he succeeded to an unusual degree. During his experience as Machinery Salesman a competitor once said: "I have never heard any one, anywhere, question Jim Dousman's honesty, and for energy he has anybody beaten I have ever seen."

Mr. Dousman died at his home in Kansas City on January 5, 1925. He is survived by his widow, Ina M. Dousman, two sons, Donald J., of Kansas City, and Albert H., of Long Island, New York.

Mr. Dousman was elected a Member of the American Society of Civil Engineers on April 3, 1922.

JOHN STANTON ELY, M. Am. Soc. C. E.*

DIED MARCH 30, 1928.

John Stanton Ely, the son of the Rev. John Hugh and Mary (Stanton) Ely, was born March 12, 1874, at Hillsboro, Ohio. He was the grandnephew of Edward M. Stanton, Abraham Lincoln's Secretary of War.

Mr. Ely was educated at Kenyon College and the Ohio Military Institute, from which he was graduated in 1892. From 1892 to 1895 he was engaged in private practice with a civil engineer in Cincinnati, Ohio, after which he was employed by the Cleveland, Cincinnati, Chicago and St. Louis Railway Company ("Big Four") as Assistant Engineer, Maintenance of Way.

During the Spanish-American War Mr. Ely served five months in Cuba with the 2d Regiment, U. S. Engineers. Directly following his return from Cuba he was engaged with the Baltimore and Ohio Railway Company in charge of a field party. He then spent a short time with the Niagara Falls Power and Light Company on construction work.

In 1901, he entered the employ of Westinghouse, Church, Kerr, and Company, Pittsburgh, Pa., and was engaged on the construction of a high-speed electric line between Scranton and Wilkes-Barre, Pa. The following year he entered the service of the Bureau of Filtration of the City of Philadelphia, Pa., which was later combined with the Bureau of Water. He remained in this position until 1918, when he was granted a leave of absence to join the New York Shipbuilding Corporation.

During this period Mr. Ely was engaged in detail construction work on the Torresdale Filters and large distribution pipe lines, serving also as Resident Engineer in charge of construction of the Torresdale preliminary filters and the design and construction of the Queen Lane Filter Plant. For the New York Shipbuilding Corporation he was Estimating, Distributing, and Construction Engineer. His duties consisted principally of estimating the progress on ships, designing buildings, as well as computing the cost of the new $11 000 000 shipyard.

In August, 1920, he returned to the Bureau of Water of Philadelphia as Engineer of Design and Construction. The most important parts of this work were the studies in relation to the water supply of Philadelphia and the proposed new sources; the construction of a 48-in. pipe line under the Girard Avenue Bridge; and the design and construction of the New Fairmount Dam, a concrete ogee dam, 1 120 ft. long, across the Schuylkill River just below the old wooden Fairmount Dam.

In August, 1926, Mr. Ely resigned to become Division Engineer of the Water Department of Newark, N. J., which position he held until February, 1928, when he was appointed Chief of the Bureau of Water of Philadelphia. His death occurred on March 30, 1928, before he actually assumed the duties of this position.

In November, 1927, he was injured in an automobile accident while on an inspection trip for the Newark water supply in the hills of Northern New

* Memoir prepared by Lyle L. Jenne, Assoc. M. Am. Soc. C. E.

Jersey. A car turning sharply across the road forced his driver to skirt the ditch, throwing Mr. Ely from the seat, striking his spine. For a time it was thought his back was broken, but he recovered from the injury. It is believed, however, that the shock seriously affected his health. In February, 1928, he went to Florida, hoping the change would benefit him. He went to the Halifax Hospital near Daytona, but became gradually worse. After a month he came north to the University Hospital in Philadelphia. In his weakened condition pneumonia developed, and he lived only five days.

Mr. Ely was of a quiet nature and possessed the quality of inspiring the respect and affection of his associates and friends by whom he is sincerely missed. He was fond of sports and was an enthusiastic and able golfer.

On October 17, 1900, he was married to Mary Elizabeth Johnson, of Louisville, Ky., who survives him.

Mr. Ely was elected an Associate Member of the American Society of Civil Engineers on March 2, 1904, and a Member on October 1, 1913.

CHARLES FREDERICK WILSON FELT, M. Am. Soc. C. E.*

DIED FEBRUARY 4, 1928.

Charles Frederick Wilson Felt was born on April 29, 1864, at Salem, Mass. He was a descendant of George Felt, who, with John Endicott, came to Salem in September, 1628, and founded, on June 24, 1629, the Town of Charlestown (now a part of Boston), Mass., named in honor of Charles the First.

The history of New England records the activities of this family and its branches in the settlement and development of the region centering around Casco Bay and Boston. One of Mr. Felt's antecedents was Captain John Felt, who bore an honorable part in the War of Independence against the British, particularly against the latter's aggression at North River Bridge, Salem, wherein they demanded the store of cannon held by the Colonists, on February 26, 1775, culminating in a melee, "which may be justly recorded the first blood of the Revolution."

Charles Wilson Felt, the father, following his graduation from Bridgewater Normal School in 1859, spent many years on the invention of a typesetting machine. Later, he became an author and publisher in Worcester, Mass. He married Martha Seeth Ropes and three children were born to this union.

The early years of Charles Frederick Wilson Felt were spent with relatives at Salem, and at Elizabeth, N. J., because of the death of his mother. Later, he resided at Danvers, Wakefield, Harvard, and Northborough, Mass., with his father, who had re-married. As a young man he worked on farms near Southborough and Bolton, Mass., attending neighboring schools, supplementing family funds with his own efforts, and helping to pay his way through college. He was graduated with the degree of Bachelor of Science from the Massachusetts College of Agriculture at Amherst in 1886.

Work was then his desire and necessity. In September, 1886, he turned to the West where an era of railway construction beckoned. He found his first position as an Axeman on the Atchison, Topeka and Santa Fé Railway, which at that time had an active program of expansion under way in the States of Kansas and Colorado. In this connection, and, subsequently, while with the Denver and Rio Grande, Arizona and Eastern, and the, then, Santa Fé line from Nogales to Guaymas, Mexico, he held the various positions that fell to a young engineer engaged in pioneer railway location and construction.

In May, 1890, he returned to the Santa Fé Company as Resident Engineer at Galveston, Tex., where he remained until July, 1892. Following a brief service on the Rio Grande Southern Railway in Colorado as Office Engineer, he again returned to the Gulf Lines in February, 1893, as Division Engineer, and in May, 1893, he was appointed Resident Engineer of the same line at Galveston. While thus employed, the Texas Railroad Commission, under the so-called Stock and Bond Law, made a valuation of the Gulf Lines of the Santa Fé Railway in which his work, on behalf of the Railway Company, was a conspicuous part.

* Memoir prepared by a Joint Committee of the Society and the Western Society of Engineers, consisting of G. W. Harris and C. A. Morse, Members, Am. Soc. C. E., and J. de N. Macomb, Assoc. M. Am. Soc. C. E.

In September, 1896, Mr. Felt was appointed Chief Engineer of the Gulf, Colorado, and Santa Fé Railway, with headquarters at Galveston. Shortly thereafter, the Company undertook an extensive program of reconstruction and improvement work, which was planned and executed under his direction. Also, under his supervision, several hundred miles of extensions were constructed in the States of Texas, Oklahoma, and Louisiana. During this time, he also directed the preparation of several important valuations, the need for which arose in connection with the demands of the Railway Commissions of the States of Texas and Oklahoma, as well as the needs of the Company, in connection with rate-case litigation.

In November, 1909, Mr. Felt was appointed Chief Engineer of the Atchison, Topeka and Santa Fé Railway, with headquarters at Topeka, Kans. This placed him in charge of directing the engineering features in connection with maintenance and improvement work on the Eastern and Western Lines of the Santa Fé System. The mileage of second track was extended; terminals and shops were enlarged; bridges were strengthened; and the physical plant of the road was generally improved. He also directed and participated in the work of preparing several important rate-case valuations.

In April, 1913, he was appointed Chief Engineer of the entire Santa Fé System, with headquarters at Chicago, Ill. He brought to this last assignment an ability enriched with experience and a high degree of engineering skill. In this position he had charge of all engineering, construction, and valuation work. This embraced the standardization of methods and plans, the use and service of materials and appliances, the development of timber treatment, the location and construction of extensions to existing lines, and the general supervision of engineering matters on the entire system. The extensions and new lines added to the System amounted to more than 800 miles.

During this time, also, there was initiated and completed the inventory, pricing, conferences, and hearings arising out of the Federal Valuation Act. Mr. Felt's special qualifications and zeal peculiarly fitted him to co-ordinate and direct this work. It is worthy of recital that when he first entered its employ, the Santa Fé operated 5 348 miles of road, which, by the close of his service, had increased to 12 349 miles of main line and branch lines and 1 724 miles of second track, to which increase he had given his best efforts—his all.

Mr. Felt was a Member and a Past Vice-President of the Western Society of Engineers. He was a Charter Member and Past-President of the American Railway Engineering Association, in the work of which he took an active part. He was also a member of the American Railway Association, American Railway Guild, and Past-President of the Chicago Engineers' Club.

He was affiliated with, and was Past-President of the Trustees of, the Hyde Park Presbyterian Church, in Chicago, at the same time retaining his membership in the First Presbyterian Church, which he joined in December, 1893, at Galveston, where the funeral and interment were held.

The following is quoted from a resolution of the Board of Directors of the Atchison, Topeka and Santa Fé Railway Company, passed at its meeting on February 7, 1928:

"*Resolved*, That the Directors express their great sorrow in the death on February 4, 1928, of C. F. W. Felt, System Chief Engineer. * * * Mr. Felt's engineering experience included surveys, location, construction, maintenance, and all engineering problems involved in operation. His ability and his thorough knowledge of engineering, and broad experience, his kindliness to his associates and his fairness in dealing with public authorities, won for him the highest respect and consideration of all. Every one who knew Mr. Felt loved him and feels in his death a personal loss."

He was married on April 6, 1904, to Clara C. Root, of Galveston, who survives him. He is also survived by one brother, Dr. E. Porter Felt, of Nassau, N. Y., two half-brothers, George H. Felt, of Northborough, Mass., and Benjamin F. Felt, of Melrose, Mass., and a half-sister, Elizabeth Ann Felt, of Boston, Mass.

Mr. Felt was elected a Member of the American Society of Civil Engineers on March 3, 1897.

JOSIAH FOSTER FLAGG, M. Am. Soc. C. E.*

DIED APRIL 13, 1928.

Josiah Foster Flagg was born in Dedham, Mass., on September 4, 1835, the son of Josiah Foster and Mary (Wait) Flagg.

From the Boston Latin School he entered Harvard University, at Cambridge, Mass., from which he was graduated in 1854 with the degree of Bachelor of Science, *Summa cum laude*. Immediately following his graduation he was employed as Rodman and Draftsman on the Cleveland, Columbus and Cincinnati Railroad.

In the spring of 1855, Mr. Flagg returned to Boston, Mass., where he was employed as Assistant Observer and Computer at the Harvard Observatory, having prepared himself for such work before graduating from college. While engaged in astronomical observations he went on one of the expeditions to England, in charge of fifty-two chronometers, for the purpose of checking the difference in longitude between Greenwich and Washington. This engagement led to his appointment as Assistant to Lieutenant Gisliss, at the Washington Observatory, in reducing the astronomical observations which the latter had previously made in Chile.

In the spring of 1857 Mr. Flagg resigned from the Observatory Staff to become Assistant Engineer under General (then Captain) M. C. Meigs, in charge of the United States Capitol and General Post Office extensions, and also of the Washington Aqueduct. He continued in this position until the close of 1859 when deficient appropriations necessitated a general reduction of the staff.

In the spring of 1860 he became Assistant Engineer to the late James B. Francis, Past-President and Hon. M. Am. Soc. C. E., in charge of the locks and canals at Lowell, Mass. This work, however, terminated toward the close of the year.

When the Civil War commenced Mr. Flagg offered his services to the nation in a civil engineering capacity. Not being successful in his patriotic efforts in Massachusetts, he went to Washington to further his claim for service. While he was there, the Battle of Bull Run occurred and in the tense situation that followed, further effort seemed hopeless.

In August, 1861, he accepted a position as Draftsman with Bement and Dougherty in Philadelphia, Pa. At that time this firm was second in importance to that of William Sellers and Company as manufacturers of machine tools. Later in the same year he was sent by this firm to the Springfield Armory to study the methods of making muskets, machine tools for quantity production of small arms being in great demand at that time.

In March, 1862, Mr. Flagg resigned this position to join the Atlantic and Great Western Railroad Company, subsequently the New York, Pennsylvania and Ohio Railroad, at Meadville, Pa. For ten years he served this Company in an engineering capacity as well as Land and Tax Agent and sometimes as Paymaster.

* Memoir prepared by James R. Chapman, M. Am. Soc. C. E.

In March, 1872, he resigned this position to take charge of the erection of machinery for an extensive nitrate plant at Iquique, Peru. This work required two years for completion. In 1874, Mr. Flagg assisted in the examination of the Magdalena River in Venezuela in connection with a project to improve it for navigation.

Following his return to the United States in 1875, he was one of three mechanical experts engaged to test steam engines and other machines entered in competition at the Cincinnati Exposition. The tests and compilation of the reports took several months.

In January, 1876, Mr. Flagg returned to Meadville, Pa., as City Engineer, taking up the private practice in his profession at the same time. He occupied this position for five years, resigning in 1881 to take charge of the construction of the Pacific end of the Mexican National Railway, with headquarters at Colima, Mexico. During the following three years he located, constructed, and operated 130 miles of line mostly in a very rough and difficult country. The fact that the office of the General Manager was in New York and that of the Chief Engineer in the City of Mexico—a ten-day journey—left Mr. Flagg in a position of great responsibility. All construction material and rolling stock were received by vessel. Right-of-way details and the disbursement of all funds were in his charge. Due to financial difficulties which caused a suspension of construction, he resigned in 1884.

After some months spent in travel, Mr. Flagg went to Cuba in the fall of 1885, to report on proposed plans for water supplies for the Cities of Santiago and Guantanamo. After returning from Cuba he was placed in charge of the up-town works of the New York Steam Company, in New York, N. Y.

Subsequently, Mr. Flagg was employed by the Engineering Commission appointed to formulate plans for sewage disposal at Washington, D. C. He was in charge of the investigations and computations necessary for the prosecution of the undertaking.

At the conclusion of this service he traveled extensively in Europe and North Africa, making many investigations relative to sewage disposal as developed in England and on the Continent. The latter years of his life were spent in retirement in Santa Barbara, Calif., where he died.

Mr. Flagg was married on April 13, 1858, to Emma A. Wiggin who died in May, 1888. One daughter was born to them, Helen B. Flagg, who died in 1902. On December 3, 1903, he was married to Florence N. Dukes, of Brooklyn, N. Y., who survives him.

Mr. Flagg was elected a Member of the American Society of Civil Engineers on October 7, 1874.

AUSTIN BRADSTREET FLETCHER, M. Am. Soc. C. E.*

DIED MARCH 8, 1928.

Austin Bradstreet Fletcher was born at Cambridge, Mass., on January 19, 1872. His father, Ruel H. Fletcher was, during a long and active life, a teacher in the public schools of Cambridge. His mother was Rebecca C. (Wyman) Fletcher. They were descendants of English stock, whose ancestors came to Massachusetts in early Colonial times.

Mr. Fletcher prepared for Harvard University at the Cambridge High School, and entered the Lawrence Scientific School in the fall of 1889. After two years of work in the general Science Course, he took courses in Civil Engineering, and was graduated in 1893, receiving the degree of Bachelor of Science.

Immediately after his graduation from college, he took a position as Secretary with the newly organized State Highway Commission of Massachusetts, and remained in the employ of this Commission until 1910. He had not been long in active work before he gave evidence of that exceptional executive and administrative ability for which he became noted. At the time he left the Massachusetts Highway Commission he was performing the duties of Executive Officer and Chief Engineer of the Commission.

The State Highway work in Massachusetts was pioneer work in this field, and attracted nation-wide attention, both for the actual work accomplished, as well as for the organization developed. Great credit was given Mr. Fletcher for the smooth-running, efficient organization that he perfected, the personnel of which was maintained for more than twenty years. This alone presents a great contrast to the many changes which have so often characterized public works enterprises.

During the time he served the Massachusetts Highway Commission, this organization was given supervision over the long-distance telephone lines along the public highways, which in consequence greatly broadened the scope of his duties.

At this period the character of highway traffic was undergoing a great change. It was evident that the past methods of construction would be insufficient to withstand motor traffic. The great problem presented was how most economically to treat the roads already built in which large sums had been invested, and also how best to build the new roads. No one foresaw these changes and the new demands better, or felt the responsibility more keenly, than did Mr. Fletcher. The many experiments and methods of treating road surfaces that were carried on by the Massachusetts Highway Commission attracted the widest attention. At the time Mr. Fletcher severed his connection with the Commission he was established as the foremost engineer in the highway field.

In 1910 he was induced to accept the position of Chief Engineer of the Highway Commission of San Diego County, California, where a construction

* Memoir prepared by A. N. Johnson, M. Am. Soc. C. E.

program involving millions of dollars was planned. He was engaged in this work scarcely a year when he was appointed the first State Highway Engineer of California. Here, he was destined to accomplish the greatest and most comprehensive program that has fallen to the lot of any highway engineer. One of Mr. Fletcher's colleagues in this work pays the following well-deserved tribute:

"So well did he install system in California that few changes have been made, or perhaps can be made, in the engineering principles and standards devised by this far-seeing pioneer of highway development. His ideas and ideals will ever remain foundation stones of the organization structure of the State Highway Department of California".

While engaged as State Highway Engineer of California, Mr. Fletcher was called in consultation, on several occasions, by the United States Bureau of Public Roads. During the summer of 1916, he assisted in drafting the rules and regulations to carry into effect the Federal Aid Road Law, which had just been passed by Congress.

Following the death, in 1918, of Logan Waller Page, M. Am. Soc. C. E., Chief of the United States Bureau of Public Roads, Secretary Houston, of the Department of Agriculture, tendered this position to Mr. Fletcher, who declined it, preferring to carry on the great road program in California, in which he was engaged.

From 1917 to 1923, in addition to his highway duties, he was President of the California State Reclamation Board, and from 1921 to 1923, he was Director of Public Works of California. In January, 1923, he was called East to act as Consulting Engineer for the New England Rail Committee. This Committee was made up of representatives of six New England States to study the whole transportation system in New England. Mr. Fletcher made a study and submitted a report on the motor-truck transportation phase of this investigation.

From September 1, 1923, until his death, Mr. Fletcher was connected with the United States Bureau of Public Roads as Consulting Highway Engineer.

In 1908, he was sent, as delegate from Massachusetts, to the First International Road Congress, held in Paris, France. He attended the Third International Road Congress held in London, England, in 1913, as delegate from California. In 1914 he was chosen President of the American Road Congress, which was held at Atlanta, Ga., in November of that year. During the World War he was a member of the Executive Committee of the California Council of Defense.

He was the author of a *Bulletin* on Macadam Roads issued by the United States Department of Agriculture in 1906. He contributed the chapter on "Drainage" in the American Highway Engineer's Handbook, and published many papers in various technical journals.

Mr. Fletcher was a member of the American Society for Testing Materials, the American Association of Engineers, the American Concrete Institute, the Boston Society of Civil Engineers, the Massachusetts Highway Association, the American Road Builders' Association, the American Association of State

Highway Officials, the Permanent International Association of Road Congresses, and a Fellow of the American Geographic Society.

He took a great interest in and was a member of a number of historical and patriotic societies, among which were the New England Historic-Genealogical Society, the Massachusetts Society of Sons of the American Revolution, and the Society of the Colonial Wars of California. He was a member of the Cosmos Club, of Washington, D. C., of the Harvard Club, of San Francisco, Calif., and of the Harvard Engineers Club, of New York, N. Y.

Mr. Fletcher died on March 8, 1928, at his home in Chevy Chase, Md. He had just returned from a business trip to Cleveland, Ohio, where he contracted a severe cold that developed into pneumonia.

On March 1, 1894, he was married to Ethel Hovey, of Cambridge. There were two children, Dorothy, now Mrs. Laurence H. Chapman of Sacramento, Calif., and Norman, who died when but seven years of age.

Although Mr. Fletcher was shy, sensitive, and modest to a fault, he was, nevertheless, an exacting and firm administrator. He wasted little time on what he considered useless argument and speculation; he was not to be stampeded. He had that essential quality of all great executives, the ability to select a capable personnel, and to inspire and maintain in his organization that spirit of loyalty and enthusiasm aptly called *esprit de corps*. While he was severely just in his dealings with subordinates, his disciplinary measures were always tempered with a kindliness that left no heart-burnings.

Those of the organization which he perfected in California speak of him with the greatest respect and loyalty. In the April, 1928, number of the Official Journal of the Department of Public Works of California, dedicated to him, there is paid this testimonial to his all too short, but useful, life:

"Mr. Fletcher lived to see the tree of his life mature and fruit in an added happiness and an enlarged usefulness given to the whole people of California. What greater monument could any one build? What greater reward could any one ask?"

Mr. Fletcher was elected a Member of the American Society of Civil Engineers on June 1, 1909.

GEORGE WASHINGTON GOETHALS, M. Am. Soc. C. E.*

DIED JANUARY 21, 1928.

George Washington Goethals was born on June 29, 1858, in Brooklyn, N. Y., the son of John Louis Goethals. His family was of Dutch extraction. He entered the College of the City of New York at the age of fourteen, and remained there for three years. He was then selected for a cadetship at the United States Military Academy by the Hon. S. S. Cox, Representative in Congress of the 6th District of New York. At West Point, Mr. Goethals was much liked by his classmates and other cadets, and maintained a prominent standing in his studies and his military duties. He was a Cadet Captain in his first-class year, and was elected President of his Class. Upon graduation in 1880, he was one of two to receive commissions in the Corps of Engineers of the United States Army.

After serving for two years at the Engineer School and with the Battalion of Engineers at Willets Point, N. Y. he was appointed Engineer Officer of the Department of the Columbia, with station at Vancouver Barracks, Vancouver, Wash. He then served for a year as Assistant to the late Col. W. E. Merrill, Corps of Engineers, U. S. Army, M. Am. Soc. C. E., on the works of improvement of the Ohio and tributary rivers. From 1885 to 1889 he was Instructor and Assistant Professor of Civil and Military Engineering at the United States Military Academy, and from 1889 to 1891, Assistant to the late Col. J. W. Barlow, Corps of Engineers, U. S. Army, on the work of improvement of the Tennessee and Cumberland Rivers.

During the next three years Captain Goethals was in charge of the Tennessee River improvement and the completion of the Muscle Shoals Canal, including the design and construction of the Colbert Shoals Lock. There, he added to his previous insight into problems of canal construction and of administration and direction of work executed by the Government without intermediary contractor.

From 1894 to 1898 he was Assistant in the office of the Chief of Engineers, U. S. Army, in Washington, D. C. At the outbreak of the Spanish-American War he was appointed Lieutenant Colonel of Engineers in the Volunteer Service and assigned as Chief Engineer of the First Army Corps. As such, he organized and constructed the water supply system of the camp at Chickamauga, Ga., and directed the engineering operations of the Corps in the Porto Rican Expedition.

At the conclusion of the war he reverted to his regular rank of Captain and was appointed Instructor of Practical Military Engineering at West Point, being also in charge of completing the water supply system of the Post and the reconstruction of the Library Building. He was promoted to a Majority in February, 1900, and served in charge of works of river and harbor improvement and fortification in Rhode Island and Southeastern Massa-

* Memoir prepared by H. F. Hodges, M. Am. Soc. C. E.

chusetts, until the creation of the General Staff of the Army under President Roosevelt and Secretary of War Root, in 1903. Major Goethals' outstanding qualifications prompted his selection on the first list of this body of picked men. His tour of duty with the General Staff had almost reached the statutory limit of four years when, as a Lieutenant-Colonel, he was selected by President Roosevelt to be Chairman and Chief Engineer of the Isthmian Canal Commission, succeeding John F. Stevens, Past-President and Hon. M. Am. Soc. C. E., who had tendered his resignation. At the same time radical changes were made in the remaining personnel of the Commission, and all the members were required to live on the Isthmus of Panama in immediate touch with the field work.

The year ended December 1, 1906, was stated by the then existing Commission in its last Annual Report to have virtually closed the stage of preparation. In that stage, plant for excavating and other work had been purchased or ordered on liberal lines, commensurate with the extent of the project. A railroad transportation system had been organized which was ample and well suited to dispose of the spoil from Culebra Cut. A good plan of recruitment of labor had been put into operation and an adequate supply assured. In matters of sanitation, quarters, and subsistence, a great deal had been accomplished, and wholesome living conditions had been created for the large and increasing forces. Good progress had been made with the preliminary work upon which the final designs for the canal structures, as then planned, could be based.

By the time the new Commission, with Colonel Goethals at its head, took charge, on April 1, 1907, the excavation was well started and a considerable volume had been removed from the canal prism in Culebra Cut and from the site of the Gatun Locks. Nevertheless, serious and difficult problems of plan, organization, and execution, vital to the success of the work, remained to be solved. The designs for the locks, dams, and accessories had to be adopted definitely and pushed to rapid completion; the manner of carrying out the work, whether by contract or hired labor, a question which was still open, had to be decided; the plant to mix and lay millions of yards of concrete had to be designed and built; certain matters which were considered as settled, such as the width of the Culebra Cut, the position of the locks and dams near the Pacific Terminal, the width of the locks, the location of the new Panama Railroad, and the position of the Atlantic breakwaters, had to be re-opened and the original decisions changed. The great terminal piers, the Pacific dry dock, the coaling plants, and the permanent shops, were so far in the future that they had not even been included in the plan; the actual work of excavation and construction had to be carried on and completed; the constructing organization changed from time to time to suit the advancing work; and, finally, an organization had to be developed for operating the canal and its subsidiary activities after completion.

Whereas the chief credit for the construction of the Panama Canal rightly belongs to the man who was for seven years its Chief Engineer and under

whom the greater part of the work was done, the Canal itself will always be a monument to the genius and ability of the American engineer. Many members of the Society, and many others whose names do not appear on its rolls, contributed to the successful result and have a share in the honor.

By Executive Order of January 6, 1908, President Roosevelt restated the duties of the Commission, placing an increased measure of authority and responsibility in the hands of the Chairman individually; and from that time forward no one doubted who was in control of the work, and no one in the Canal force doubted its ultimate success.

The achievements of the next seven years are already well-known to those who will read this memoir, and require no extended recounting here. The Canal was finished within the estimate of cost made in 1908, and was opened to traffic before the date then set. Operation, however, was interrupted later for some months by slides in the Culebra Cut, but the Canal has since been serving a heavy traffic which threatens to tax its capacity.

Toward the conclusion of the construction period, when the organization for operation was developed, Colonel Goethals was made the first Governor of The Panama Canal. On March 4, 1915, he was made a Major General of the Line of the Army and received the thanks of Congress for his services. On November 1, 1916, he was retired from active service at his own request.

In 1917 he was appointed State Engineer of New Jersey by Governor Edge. After a few months spent in laying out a new highway system for New Jersey and in practice as Consulting Engineer, he was appointed by President Wilson to be General Manager of the Emergency Fleet Corporation, United States Shipping Board. He was pronounced in his opposition to the plans for extensive construction of wooden ships, and resigned from the Fleet Corporation in July, 1917, to resume his consulting practice.

In December, 1917, General Goethals was recalled to active service and made Acting Quartermaster General, United States Army. In this capacity and as Assistant Chief of Staff and Director of Purchase, Storage, and Traffic, he served through the remainder of the World War, and until March 4, 1919, when he was relieved at his own request from further active duty. He then again took up his practice as Consulting Engineer in New York City, a practice in which he continued until his death. During its course he was associated for a time with the firm of Jamieson, Houston, Graham, and Jay, subsequently absorbed by George W. Goethals, Inc., of which Company he was President. He was also President of Goethals, Wilford, and Boyd, Inc., and of the American Ship and Commerce Corporation.

General Goethals and his associates were consultants on many important works, such as the Inner Harbor Navigation Canal at New Orleans, La.; the Columbia Basin Irrigation Project; the East Bay Municipal Utility District, of Oakland, Calif. and the Lake Worth Inlet District, in Florida. The New York-New Jersey Port and Harbor Development Commission, created by Acts of the State Legislatures in 1917, selected him as its Chief Consulting Engineer, and he continued as such with The Port of New York Authority created by the same Legislatures in 1921 to carry into effect the compre-

hensive plan prepared by the earlier Commission for re-organizing the facilities for, and modernizing the methods of, handling the commerce of the Port of New York. His connection with this great work continued until his death. In January, 1923, he was appointed Fuel Administrator of the State of New York, and served as such for a few months.

General Goethals possessed in a marked degree the technical qualifications, the good professional judgment, and the administrative and organizing ability which characterize the great engineer; but perhaps the most effective weapon in his armament was his tremendous driving power. This was not shown in arbitrary pressure or nagging criticism. Indeed, it would not be far wrong to state that it acted as much by inspiring as by spurring his subordinates; for he spared himself no more than he did them.

He succeeded in winning the confidence, respect, and even affection of the working force on the Isthmus, to a remarkable extent. Every one among them felt sure of fair treatment at his hands. His spirit rose especially against anything which savored of oppression of the lower by the higher; this characteristic was so well-known that all felt free to lay any case of real or fancied injustice before him in person, without fear of unpleasant after effects. His Sunday mornings were spent in such personal interviews. While they thus absorbed much of his scanty leisure time, and even more of his nervous energy and, perhaps, his patience, they were more than worth while, because of the intimate relation which they built up with the men in the ranks, and the consequent abiding trust in him which was the certain result.

General Goethals was a warm and steadfast friend to those to whom he gave his confidence; but this was not given readily, nor until he had proved to his satisfaction that it could be based upon esteem as well as personal regard. Once given, it was rarely withdrawn. Those who possessed it would have gone to any lengths for him.

He was a man of strong opinions, but open to argument and tolerant of difference from his own views, even of opposition to them, as long as a plan might be still under consideration. Once the matter had been decided and directions given, he expected loyal compliance, and did not welcome any attempt to revive the issue. To take such a step one had to have a very good reason and be sure of his ground.

For his service during the World War General Goethals received the Distinguished Service Medal from the President of the United States; the Cross of Commander of the Legion of Honor from the President of the French Republic; and was made Honorary Knight Commander of the Order of St. Michael and St. George by the King of England. He received also the Grand Cordon of the Order of the Striped Tiger, 2d Class, from the Chinese Government. He was the holder of numerous medals, among them the John Fritz Medal "for achievement as builder of the Panama Canal"; the medal of the National Geographic Society; the Cullum Medal of the American Geographic Society of New York; the President's Medal of the Architectural League; and the medal of the National Academy of Sciences. He received Honorary Degrees of Doctor of Laws from Harvard, Yale, and Princeton

Universities, the University of Pennsylvania, Johns Hopkins University, and Dartmouth College; of Doctor of Philosophy from the Chicago Polytechnic Institute; of Doctor of Science from Columbia and Rutgers Universities; and of Bachelor of Science from the College of the City of New York, which he attended before going to West Point. He was a member, regular or honorary, of many clubs and societies, social and technical, including the Institution of Civil Engineers of Great Britain and the Royal Engineering Society of Holland.

General Goethals' death was the occasion of many tributes to the high regard in which he was held. The President of the United States wrote that,

"General Goethals will be remembered most widely for his great achievement in the construction of the Panama Canal, as Chief Engineer, and in placing its operation and the administration of the Canal Zone on the extraordinary basis of efficiency which has made it so successful. But his skill and genius were no less marked in other work. Particularly, were they invaluable to the Government during the World War when, coming back from well-earned retirement, he displayed his great ability in a variety of activities. His name was known throughout the world and will remain for all time on the roster of those who have done big things for our country."

The Governor of the State of New York wrote that in his passing,

"The State of New York, as well as the nation, lost a notable public servant. He brought to his duties as Chief Consulting Engineer of The Port of New York Authority that same expert wisdom which made him the genius of the Panama Canal.

"At great personal sacrifice he responded to my request to become Fuel Administrator of the State at a time when we were suffering a serious fuel emergency * * *. Here again, with his great executive talent, his power of decision and of direction, he gave to the State a useful and successful service.

"He has left behind a permanent memory of great personality and a splendid record of achievement."

Many other expressions from those who had been associated with him show how deeply they were impressed by his exceptional ability and by the strength of his character, the purity of his ideals, and the integrity of his motives. In the words of one of them, "He was at all times found equal to the important and responsible positions in which he was placed."

General Goethals was married in 1884 to Effie Rodman, of New Bedford, Mass. His widow and two sons, George R. Goethals, M. Am. Soc. C. E., and Dr. Thomas R. Goethals, of Brookline, Mass., survive him.

General Goethals was elected a Member of the American Society of Civil Engineers on March 1, 1910, and served as Director in 1918.

ALVAH SEYMOUR GOING, M. Am. Soc. C. E.*

DIED MARCH 21, 1927.

Alvah Seymour Going was born at Portland, Ore., on April 7, 1860, the son of James Wallace and Isabella (Eads) Going. His father's family settled at Shirley, Mass., early in the Eighteenth Century, the family name then being Gowing. Mr. Going was educated in the public schools of his native city and for a time attended the Oregon State College.

In 1881 he began his railroad career as Levelman on location for the Oregon Railway and Navigation Company. From 1882 until 1883 he was employed as Transitman on construction of the Clark's Fork Division of the Northern Pacific Railway, and from March to October, 1883, he was in charge of exploration and location surveys for the Astoria and Forest Grove Railroad.

At this time the West was rapidly developing; many railroad projects were under way and this character of work fascinated Mr. Going. He was engaged on many of these projects in Oregon, Washington, Idaho, and in British Columbia, Canada, from 1881 to 1890, when he formed a partnership with Mr. A. W. Miller and entered the general practice of engineering at Port Townsend, Wash. During his association with Mr. Miller he served as Engineer of several small railroads then under construction and also for the Victoria, Port Crescent and Chehalis Railroad, a line about 150 miles in length.

In 1891 Mr. Going moved to Victoria, B. C., Canada, where he engaged in general engineering practice but specialized in the exploration and location of railroads. This practice included work on many small mining railroads and for a time he was engaged on collieries on Vancouver Island.

In 1903 Mr. Going was employed as Reconnaissance Engineer for the projected Grand Trunk Pacific Railway, to explore various routes through the Rocky Mountains, in advance of location survey parties. On these expeditions he was obliged to travel alone, except for Indian guides whom he employed from time to time. This exploration work was started late in 1903, making it necessary for him to spend most of a severe winter alone in a hostile, uninhabited country. He started this expedition with two horses, but was obliged to abandon them and complete the work on foot, arriving at Fort George, B. C., Canada, after many hardships, but in possession of much invaluable information regarding the passes through the Continental Divide. Few engineers could have endured the hardships of this work. It was in this class of occupation that he found the greatest interest. He was at his best when confronted with the problem of finding a location for a railroad through new territory.

In October, 1905, Mr. Going was engaged by the Minneapolis and St. Louis Railroad Company in South Dakota, where he located and built 130 miles of line from Conde to LeBeau on the Missouri River.

During 1907 he became associated with the Grand Trunk Railway Company of Canada as Locating Engineer, with headquarters at Montreal, Que.

* Memoir prepared by R. D. Garner, Esq., Providence, R. I.

As such, he supervised all the location work under way at that time, notably the 85-mile extension of the Central Vermont Railway to Providence, R. I. When the Grand Trunk Railway and the Canadian Northern Railway were merged into the Canadian National Railways, Mr. Going was retained in the Bureau of Economics as Terminal Engineer. He held this position until his death.

Mr. Going is best remembered by those who were fortunate enough to be associated with him for his unfailing energy in helping younger engineers. He was never too busy or too tired to lay aside the work in hand to aid and counsel others. He possessed a remarkable memory which was combined with an almost uncanny ability to know exactly where to look for the engineering information that might be wanted. He was an omnivorous reader. No technical publication escaped him and from these he made innumerable notes and references for future use. In disposition, he was the personification of modesty, and ever tolerant of the defects in the characters of others.

Mr. Going was a member of the Masonic fraternity. He was also a member of the American Railway Engineering Association and served on its Committee on the Economics of Location for many years, having devoted a great deal of study to this work.

While in the South, in December, 1927, Mr. Going was taken seriously ill. He returned to Montreal, where after a lingering illness he died at the Royal Victoria Hospital on March 21, 1927. He is survived by his widow, Harriett Jackson Going, and one daughter, Mrs. Chase Going Woodhouse, of Washington, D. C.

Mr. Going was elected an Associate Member of the American Society of Civil Engineers on May 4, 1892, and a Member on June 7, 1899.

SILAS MAXWELL HAIGHT, M. Am. Soc. C. E.*

DIED SEPTEMBER 29, 1928.

Silas Maxwell Haight was born in Elmira, N. Y., on May 5, 1883. His father, Maxwell Haight, one of those high-minded, great-hearted, jovial men who endear themselves to all, enjoyed a wide popularity in his community. His mother was a woman of beautiful character and great courage whose maiden name was Frances Hendy. She was a direct descendant of John Hendy who held a commission as Colonel in General Washington's Army during the Revolutionary War, himself a surveyor and engineer. He was the first white settler in the Chemung Valley and his descendants have always lived in or near the present City of Elmira.

Mr. Haight's early education was acquired in the public schools and High School in Elmira. He afterward entered Lafayette College, at Easton, Pa., from which he was graduated in June, 1908, with the degree of Civil Engineer.

His first professional engagement was with the City of Elmira as Transitman in the Engineering Department, where he soon assumed responsible charge of much municipal work. He remained with the City of Elmira as Construction Engineer until the outbreak of the World War. In August, 1917, he entered the Officers Training Camp, at Fort Niagara, New York, where he was commissioned a First Lieutenant. He was assigned to duty overseas and attached to the 475th Aero Squadron, American Expeditionary Force. After service in France, he was ordered to Lopcombe Corner, England, where he was placed in full charge of the design and construction of airdrome buildings and pertinent works.

On his return after the war, Mr. Haight resigned his municipal position to take up private engineering work in which he was very successful. He constructed many miles of brick and concrete pavements, and was Resident Engineer during the building of the beautiful reinforced concrete bridge in Elmira which spans the Chemung River at Main Street. After many other private engineering accomplishments, Mr. Haight accepted an important position as Engineer with the Highway Products and Manufacturing Company, manufacturers of Armco corrugated iron pipe. He assumed complete charge of the design and installation on many large drainage projects, flumes, sewers, etc., and his work in iron-pipe promotion was so pronounced and successful that he soon attracted the attention of several other manufacturers who accordingly bid for his services. Just a few months before his death, he had accepted a responsible position with the Sweet Steel Company, of Williamsport, Pa., and he was taking a business trip for this Company when he was instantly killed in a motor car accident, in the prime of his busy life.

"Si" Haight, as he was familiarly called, had friends by the thousands. He was sociable and kind, with a great mind and memory. Blessed as he was with a keen sense of humor and happy disposition, he won warm friends wherever he traveled. He was a gentleman at all times, big-hearted, noble, and brave, and ever loyal to his legion of friends who mourn his loss.

* Memoir prepared by M. W. Wipfler, M. Am. Soc. C. E.

Immediately following Mr. Haight's funeral, which was largely attended, several of his more intimate friends met at the City Club in Elmira and perfected a permanent organization known as the "Si Haight Club". The members of this Club intend to meet once each year on the anniversary of Mr. Haight's death and hold a banquet in his honor, at which time fitting tributes will be extended to his memory.

Mr. Haight was a member of Union Lodge No. 95, F. and A. M., Elmira; the Corning Consistory; Kalurah Temple, A. A. O. N. M. S.; the Southern Tier Shrine Association; Elmira Lodge of Elks No. 62; Phi Delta Theta Fraternity, and various other organizations.

He was unmarried and was always devoted to his mother and to his sister, Mrs. Grace Haight Parker, who is his sole survivor.

Mr. Haight was elected a Member of the American Society of Civil Engineers on June 6, 1927.

WILLIAM BURR HARRISON, M. Am. Soc. C. E.*

DIED OCTOBER 20, 1926.

William Burr Harrison, the son of William Elsey Harrison and Elizabeth (Edwards) Harrison, was born at Leesburg, Va., on February 27, 1871. After attending High School at Leesburg, he entered the Virginia Military Institute at Lexington, Va., from which he was graduated in 1892 near the head of his class, with a Civil Engineer's degree.

After his graduation, Mr. Harrison lost no time in taking up active practice. He immediately accepted a position as Engineer in charge of construction of a three-span highway bridge across Goose Creek, at Evergreen Mills, Va. When this work was completed in March, 1893, he was employed by the Lake Shore and Michigan Southern Railway Company until June.

From June, 1893, to January, 1899, he was with the United States Engineer Office at St. Louis, Mo., in responsible charge of surveys, triangulation, precise leveling, design, and construction of river and harbor work. From January, 1899, to 1902, Mr. Harrison was with the United States Engineer Office in Washington, D. C., in responsible charge of surveying, triangulation, leveling, the laying out and the receiving of bids for contract work, and the design and making of computations for various classes of river and harbor work. With the same Department from 1902 to 1908 he was in responsible charge of most of the field work and the preparation of designs, computations, and estimates on which contractors were paid. From 1908 to 1917, he was Principal Assistant Engineer in responsible charge of all office and field work, as well as the supervision and direction of all river and harbor work under the same Office.

Mr. Harrison entered the military service of the United States in February, 1917. He was appointed a Major, Engineer Section, Reserve Corps, and as such was on active duty during the World War, in the Office of the Chief of Engineers in Washington, D. C., with the Engineer Replacement Troops at Camp Humphreys, Virginia, and, later, with the 153d Engineers at Camp Shelby, Mississippi.

In July, 1920, he was commissioned in the Regular Army and after a short period of duty in Washington and at Headquarters, 11th Corps Area, he was sent to the Canal Zone, where he remained until March, 1924. After his return to the United States, Major Harrison was on duty at Fort Hayes, Ohio, until his retirement on September 21, 1926, on account of physical disability.

By his death at Walter Reed Hospital, on October 20, 1926, his country lost a loyal soldier, an untiring and conscientious worker, and a citizen of the highest type.

* Memoir prepared by B. Powell Harrison, M. Am. Soc. C. E.

Major-General Hines, Chief of Staff, U. S. A., writing of him, stated,

"Although Major Harrison had been a member of the military establishment but a few years, his ability, earnestness, and the conscientious performance of the duties assigned him won the commendation of those with whom he was associated."

He was a member of the Protestant Episcopal Church. In December, 1906, Major Harrison was married to Lelia Virginia Wheeler who, with one daughter, survives him.

Major Harrison was elected a Member of the American Society of Civil Engineers on April 30, 1912.

HANS HELLAND, M. Am. Soc. C. E.*

DIED MARCH 9, 1928.

Hans Helland, the son of Hans and Marie Helland, was born at Bergen, Norway, on August 10, 1854. He received his early education at the Primary and High Schools of Bergen and Christiania. He then attended the Technical Institute at Trondhjem, and, still later (1879), was graduated from the Royal Saxon Polytechnicum, of Dresden, Saxony, Germany, with the degree of Ingenieur Civil.

Prior to his graduation in 1879, however, Mr. Helland had served his apprenticeship in the Laxevoag Machinery Factory, and had been an Assistant Engineer on the Royal Norwegian Railways. The year following his graduation he became Statistician for the Royal Railways and the Department of Highways of Norway.

In 1881, he emigrated to America, settling in Texas, where he became imbued with the "spirit of the West", and henceforth remained to devote a useful life to transforming a frontier State to its present highly industrialized civilization. The crying need of that day was the construction of railroads, and naturally Mr. Helland assumed his share of this duty. On his arrival in Texas in 1881, he immediately became a Construction Engineer on the Texas Central Railroad and remained with that Company until 1889, when he became Vice-President and General Manager of the Central Texas and Northwestern, and the Fort Worth and New Orleans Railroad Companies. When these two lines were consolidated with the Houston and Texas Central Railroad in 1902, he became Maintenance-of-Way Engineer for the entire System (now a part of the Southern Pacific System). He remained in this capacity until 1906, when he resigned to locate and construct the Panhandle Short Line. In 1908, he became Maintenance-of-Way Engineer for the San Antonio and Aransas Pass Railroad Company, with which he remained until 1913.

From 1913 to 1921, during its first struggle toward greater improvements, the City of San Antonio, Tex., employed him as its City Engineer. It is noteworthy that, after approximately thirty-two years of active building and improvement of railways in Texas, Mr. Helland should then take up the work of enlarging a pioneer city to one of metropolitan proportions.

After 1921 he was engaged in the private practice of engineering in San Antonio, until his death, when his son Hans Helland, III, assumed the direction of the office.

Mr. Helland was a member of the American Society for Municipal Improvements and the San Antonio Scientific Society; he was also a member of the Polyteknisk Forening of Norway, and of the Ingenieur Verein of Dresden, Saxony.

Mr. Helland was elected a Member of the American Society of Civil Engineers on June 3, 1915.

* Memoir prepared by Rufus C. Thaxton, Assoc. M. Am. Soc. C. E.

KEARNEY EVERETT HENDRICKS, M. Am. Soc. C. E.*

DIED MAY 14, 1927.

Kearney Everett Hendricks, the son of George and Priscilla (Spoon) Hendricks, was born near Asheboro, N. C., on January 9, 1878. He was graduated from Guilford College, Guilford, N. C., in 1900, and from Haverford College, Haverford, Pa., in 1902. In 1903, he attended Johns Hopkins University as a graduate student in mathematics and physics.

From July, 1902, to March, 1903, Mr. Hendricks was engaged as Leveler on railroad location, and from March to October, 1903, as Transitman for the Norfolk and Western Railroad Company. From August, 1904, to April, 1905, he served as Transitman for the Carolina and Northwestern Railway Company, in North Carolina, and from April, 1905, to March, 1907, was Resident Engineer on heavy mountain construction, building a tunnel, ½ mile long, for the Clinchfield, Carolina, and Ohio Railroad Company. For a few months in 1907, he was Assistant Engineer on a small railroad in North Carolina. In September, 1908, he accepted a position as Instructor in Civil Engineering at Lehigh University, Bethlehem, Pa., where he remained for eight years.

In September, 1916, Mr. Hendricks was engaged as Assistant Engineer in charge of construction of a large earth and concrete dam for the Southern Power Company, at Bridgewater, N. C. He remained with this Company until February, 1920.

For a few months in 1920, Mr. Hendricks served as Draftsman for the Arizona State Highway Department, as well as Estimator on Federal Aid Projects, and the designer of two reinforced concrete arch bridges. From July, 1920, to January, 1921, he acted as Chief Draftsman for the Salt River Valley Water Users Association, and from that time to July, 1923, he was Assistant Engineer of the Southern Power Company. In July, 1923, he accepted a position as Engineer for the Lakeview Park, Incorporated, at Asheville, N. C., which position he held until his death on May 14, 1927.

On August 17, 1918, he was married to Ethel M. McGill, of Natural Bridge, Va., who, with one daughter, Priscilla, survives him.

Mr. Hendricks was an engineer of unusual ability and wide experience. He impressed all with whom he came in contact as being dependable, honorable, and unassuming, and he was faithful to every trust that he ever undertook.

As a child, he was affiliated with the Methodist Episcopal Church, but, later, favored the Quaker faith. He was a member of the American Association of Engineers and of the Kiwanis Club.

Mr. Hendricks was elected an Associate Member of the American Society of Civil Engineers on October 4, 1910, and a Member on June 6, 1921.

* Memoir prepared by F. H. Cothran, M. Am. Soc. C. E.

JAMES ORMEROD HEYWORTH, M. Am. Soc. C. E.*

DIED MARCH 15, 1928.

James Ormerod Heyworth was born in Chicago, Ill., on June 12, 1866, the son of James O. and Julia F. (Dimon) Heyworth. He was educated in the public schools of Chicago and was graduated from Yale University, New Haven, Conn., in 1888.

Mr. Heyworth was both an Engineer and a Contractor, a great amount of the work carried out by him as Contractor having been designed under his personal direction as well. He became interested in concrete construction early in his career and was the Contractor on some of the earlier undertakings in which reinforced concrete was used in the United States. Among other work, he designed and built the first concrete building in Chicago, in 1904.

He was always interested in efficient construction and accordingly underwent a rigorous training in the field, spending considerable time on the work, in order to keep in close personal contact with it. He was a member of the firm of Christie, Lowe, and Heyworth from 1897 to 1903, after which he operated under his own name. During the last few years his activities were carried on under the name of James O. Heyworth, Inc.

Mr. Heyworth was particularly interested in difficult and hazardous engineering undertakings, or in those in which speed of construction was one of the essential factors. Heavy excavation and difficult foundation and concrete work, railroad track elevation, bridges, hydro-electric plants, locks, dams, piers, wharves, docks, canals, tunnels, and caissons, both in the United States and in Canada, constituted the bulk of his work.

In the course of his career, Mr. Heyworth was called upon to restore and rebuild eight dams which had failed. In every case the work was carried out safely and successfully under his direction.

His desire to be engaged in difficult enterprises was displayed in the building of the concrete arch bridge over the Mississippi River between St. Paul and Minneapolis, Minn. In North Carolina, he achieved a record, not only for speed in concrete road mileage built during one season, but also for the uniform excellence of the work.

Mr. Heyworth had many inventions to his credit, among others being that of the modern dragline excavator, which was developed by him.

He was a great believer in team work and during his years of activity was assisted by a loyal organization working under his direction. He was an indefatigable worker, personally laying out details of plant and equipment on his contract work and always eager to apply new methods of merit in its prosecution. He was one of the first to use a central flexible electric power plant on large construction work and was a great believer in the use of Diesel engines for this purpose.

Mr. Heyworth's activities, however, extended beyond engineering and contracting lines. He was deeply interested in civic and philanthropic

* Memoir prepared by the following Committee of the Illinois Section: I. F. Stern, *Chairman*, A. J. Hammond, and O. E. Strehlow, Members, Am. Soc. C. E.

affairs, full of human sympathy, an enthusiastic yachtsman and fisherman, and a lover of the great outdoors. Some years ago he was President of the Local Branch of the Izaac Walton League of America and also Commodore of the Chicago Yacht Club.

He was instrumental in securing the present quarters of the Chicago Engineers Club. When the opportunity offered itself, he took an option on the property in his own name, without consulting the members of the Board, and called a meeting of the prominent members of the Club. By the time the Club membership voted to secure quarters of its own, the value of this property had risen very considerably. Mr. Heyworth then turned his option over to the Club and, as a result of the increased value of the property, there was no difficulty in floating a bond issue. During these trying times, Mr. Heyworth was induced to take the Presidency, which he held for three consecutive years at a great personal sacrifice to himself. To him, more than to any other man, credit should be given for enabling the Engineers Club to acquire its own quarters.

Mr. Heyworth was always enthusiastic in regard to every enterprise with which he was connected. He was intensely loyal to his Alma Mater, Yale University, from the day of his graduation to that of his death, and always a "prime mover" in anything pertaining to her interests. He was a member of the Executive Committee of the Yale Advisory Board; Chairman of the North Central Division of the Yale Endowment Fund; and a member of the Yale Clubs of Chicago and New York, as well as of other college organizations and clubs.

In 1917, he heeded the call to assist in war activities and accepted the position of Manager of the Division of Wood Ship Construction of the Emergency Fleet Corporation, United States Shipping Board, which he held until after the end of the World War. During this period he turned his own contracting business over to his subordinates and devoted himself entirely to the work of his Department of the Shipping Board, which, under his direction, showed a great record for efficiency. For some time, before the end of the war, fifty ships completely equipped, ready for service, were being turned out each month.

In 1902, Mr. Heyworth was married to Martica Gookin Waterman. They had two children, Francis Dimon and James O., Jr.

He was in the prime of his powers at the time of his death. He was taken ill in August, 1927, and was ordered by his doctors to Arizona with the hope that the dry climate would restore him to health. For a time he made progress toward recovery, but later complications set in and he grew steadily worse. He asked to be taken home to Chicago where he passed away a few hours after his arrival.

His death is deeply mourned, not only by his family and friends, but by a host of younger engineers and contractors who came to him for advice and help in their troubles and to whom he gave an unstinting measure of his broad training and ripe experience.

Mr. Heyworth was elected a Member of the American Society of Civil Engineers on May 6, 1914.

JOHN BYERS HOLBROOK, M. Am. Soc. C. E.*

DIED AUGUST 20, 1928.

John Byers Holbrook was born in New York, N. Y., on July 12, 1872. He was the son of Edwin W. and Fanny Chick Holbrook, both parents being descendants of families prominently identified with the early history of Manhattan. His father was engaged in commercial banking for many years, and at the time of his death was the oldest living member of the Union League Club. His son became associated with this Club on attaining his majority.

On completing his academic schooling in New York, Mr. Holbrook entered the Pennsylvania Military College, at Chester, Pa., from which he was graduated in 1892 with the degree of Civil Engineer. In 1893, he entered Cornell University, graduating in 1896 with the degree of Mechanical Engineer. At Cornell, he joined the Delta Kappa Epsilon Fraternity, with which, during his life, he maintained an active connection. He held the office of President of the Alumni Association of his Chapter and was diligent in his efforts to influence the young students in making the most of their college life.

Upon leaving college, Mr. Holbrook spent five or six years in perfecting himself in steam power plant design and operation, by employment with various companies, notably the E. W. Bliss Company, Brooklyn, N. Y., and the New York Edison Company and the American Blower Company of New York City. In 1901, he began private practice as Consulting Engineer, specializing in steam power plants and heating and ventilating. He designed and supervised the installation of the power plants in many of the earlier skycrapers of downtown New York, among which the Trinity Building, City Investing Building, and Commercial Cable Building are of importance. Later, his work extended throughout the United States and was marked by a thoroughness and conscientious attention to detail that resulted in very successful installations.

Mr. Holbrook served an enlistment of five years in Squadron A, New York National Guard. He always maintained his interest in the Veteran Organization and, in 1916, returned to active service for the duration of the World War.

His personality was characterized by a rare cheerfulness and geniality which drew to him, and held continuously, an unusual number of loyal friends.

He was married on October 3, 1903, to Blanche V. Meehan, of New York, who survives him.

Mr. Holbrook was elected a Member of the American Society of Civil Engineers on April 6, 1909.

* Memoir prepared by W. W. Ricker, Esq., New York, N. Y.

HAROLD WELLINGTON HORNE, M. Am. Soc. C. E.*

DIED MAY 23, 1928.

Harold Wellington Horne, the son of George and Mary (Barnard) Horne, was born at Belmont, Mass., on October 12, 1873. He was graduated from Harvard University with the degree of Bachelor of Arts in 1894 and from the Lawrence Scientific School, *Magna cum laude*, with the degree of Bachelor of Science in Civil Engineering in 1896.

Mr. Horne's first employment was as Rodman with the Metropolitan Water Board of Massachusetts, in September, 1896. Later, he was advanced to the position of Transitman and remained on this work until March, 1906, having been engaged on various structures for the distribution of water to Boston, Mass., and surrounding municipalities.

From March, 1906, to January, 1912, he served as an Assistant Engineer with the Board of Water Supply of the City of New York, first, on location surveys for the Catskill Aqueduct within the Hudson River Division and, later, in charge of the construction of a section of the Aqueduct which included the Bull Hill Tunnel and the Foundry Brook Pipe Siphon.

In January, 1912, Mr. Horne became associated with the Board of Water Commissioners of Hartford, Conn., in the capacity of Division Engineer on the work then being undertaken for an additional water supply. At first, he was occupied with the extensive topographical and subsoil surveys for reservoir and dam locations and title search of large areas of land takings. After this he took charge of the construction of the various structures comprised in the Nepaug Reservoir, including the large earth dam at Philips Brook, several miles of relocated highway, tunnel, and grade conduit, together with 8 miles of 42-in. pipe and the main dam, which was a high concrete structure. Later, he undertook the construction of the long earth dam at New Hartford, Conn., for the compensating reservoir. This work was concluded in March, 1918.

From March, 1918, to December, 1921, he was Assistant Division Engineer with the Miami Conservancy District of Ohio in charge of the construction of the Englewood Dam for flood control of the Miami River.

There followed an interval of about five years devoted to private affairs and certain business enterprises in which engineering played no large part. Mr. Horne, however, became convinced that permanent retirement was not wise. His education, his experience, and his native talent all combined to make engineering the most useful as well as the most congenial field for his effort. Consequently, he resumed his professional work.

In October, 1926, he was called to the Metropolitan District Water Supply Commission of Massachusetts, as Assistant District Engineer. On June 1, 1927, he was promoted to the position of District Engineer in which capacity he continued until the time of his death. He was given general charge of the preliminary surveys and sub-surface explorations for the aqueduct tunnel

* Memoir prepared by Samuel D. Dodge, Esq., Suffern, N. Y.

from the Swift River Reservoir to the Wachusett Reservoir and, subsequently, of the construction of a part of this tunnel. He lived to complete only the shafts.

To his friends and associates is left the memory of a kindly spirit, a wide culture, especially in the arts of music and literature, and a soundly educated mind. They recall, moreover, the singular fidelity and painstaking devotion to whatever task was in hand, tedious and uninspiring though it sometimes was, which were the true expression of his ingenuous and honorable nature.

Mr. Horne was married in 1915, to Anna Garfield Davis who, with two children, Richard and Mary Barnard, survives him.

He was a member of the New England Water Works Association and of the Harvard Club of New York.

Mr. Horne was elected an Associate Member of the American Society of Civil Engineers on October 2, 1907, and a Member on December 3, 1913.

DAVID JANNEY HOWELL, M. Am. Soc. C. E.*

DIED AUGUST 28, 1928.

David Janney Howell was born in Charlestown, W. Va., on September 8, 1863, the son of Samuel and Adele (LaFitte) Howell. His father was of Scotch ancestry, while his mother was of French descent.

Mr. Howell received his early education in the Charlestown Academy, Charlestown, and afterward attended Washington and Lee University, at Lexington, Va.

His early experience—starting as Rodman and afterward being promoted to Instrumentman and Topographer on railroad surveys in Virginia—was followed by his Civil Service appointment in 1884 in the United States Geological Survey as Assistant Topographer. This position he held until his resignation in 1887 to enter private practice in civil engineering in Washington, D. C.

From 1890 to 1897 Mr. Howell's practice embraced the planning, surveys for, and superintendence of, the improvements of various suburban properties within the District of Columbia and the adjoining States. He was also engaged on surveys for and superintendence of the construction of the walks, roads, lake, and bridge in the National Zoological Park; municipal improvements in Alexandria, Va., as City Engineer; surveys, construction, and superintendence of electric railways in Washington, D. C.; an 80-ft. iron span bridge and 50-ft. short-span bridge over Great Hunting Creek in Fairfax County, Virginia; and engineering work and superintendence on drainage canals in North Carolina. In addition, he acted as Chief Engineer for the Washington, Alexandria, and Mount Vernon Railway Company between Washington and Alexandria, exclusive of electrical equipment.

From 1897 to 1910 his work embraced the following positions: Assistant Engineer under the United States Board of Engineers on Deep Waterways in charge of surveys, investigations, and report of the Oswego Route, Eastern Division, from Albany to Herkimer, N. Y., about 90 miles for the ship canal from the Great Lakes to the Atlantic Ocean; Engineer to the New York State Canal Advisory Committee (appointed by President Roosevelt, then Governor of New York State), making estimates of cost and report on the enlarged Erie Canal, from Troy to Buffalo, N. Y.; Consulting Engineer to the New York State Engineer and Surveyor, Edward A. Bond, M. Am. Soc. C. E., in charge of surveys, plans, estimates, and printed report to the New York State Legislature on the enlargement of the canals of the State known as the Barge Canal, the estimated cost of which was about $100 000 000; surveys, plans, and specifications for the water-works system, at Fultonville, N. Y., and surveys and plans for the development of water power on the Shenandoah River above Harpers Ferry, W. Va., for the Shenandoah Electric Light and Power Company; Engineer to the late F. W. Hodgdon, M. Am. Soc. C. E.

* Memoir prepared by Beaudric L. Howell, Assoc. M. Am. Soc. C. E.

Mr. Howell also served as Engineer of the Massachusetts River and Harbor Commission, on plans and estimates of cost of locks on the Taunton River, Boston Ship Canal; report on improvements to water-works system at Frostburg, Md.; electric railway surveys and report, Canandaigua, N. Y., to Lake Ontario, 30 miles; the planning and construction, as part owner, under franchise of the water system and electric plant at Welch, W. Va., and the direction of its operation until it was sold in 1911; the investigation and report on additional water supply for Bristol, Va., and the preparation of plans for, and the superintendence of the construction of, a spring supply and 4 000 000-gal. reservoir; plans and report on the new distributing reservoir, Alexandria Water Company, Alexandria, Va.; railroad surveys from Cherry Run to Cumberland, Md.; investigation and full report with estimates of cost for a dam and large storage reservoir and 5½ miles of pipe line, and the purchase of land and water rights for the same, for the Alexandria Water Company, known as Barcroft Dam and Reservoir; plans for and the general supervision of the new reservoir, at Warrenton, Va.; report on improvements for the water-works system, Washington County Water Company, Hagerstown, Md.; general report and appraisal of plant, Alexandria Water Company; surveys and plans for a new dam and storage reservoir and pipe line, Charlottesville, Va.; surveys and reports on the sewerage system, Laurel, Md.; and consultation and report on an additional water supply and about 7 miles of pipe line in Bristol, Va.-Tenn.

From 1910 until 1918 Mr. Howell was engaged in the following work: Surveys for the planning of and superintendence of the construction of surface and underground improvements for the development of numerous suburban properties in the District of Columbia, Maryland, and Virginia; all engineering work and superintendence of construction of the Barcroft Dam and Reservoir for the Alexandria Water Company, the dam being 500 ft. long, with a maximum height of 70 ft., and the reservoir storage being about 600 000 000 gal.; and the investigation and report on a proposed storage dam for electric power in Virginia and various valuations and reports on water-works properties and sewerage systems.

Mr. Howell was made Manager of the Alexandria Water Company, of Alexandria, and acted as such for a period of about two years. His work with this Company embraced plans for and superintendence of its filtration plant. He was also engaged on complete surveys, plans, and superintendence for a separate sewerage system for the Town of Potomac, Va.; and a complete report, with estimates of cost, for a water supply and sewerage system for Arlington County, Virginia, embracing about 30 sq. miles.

During 1918 and 1919, Mr. Howell served as Supervising Engineer in the United States Quartermaster Department, in charge of surveys, plans, and the construction of surface and underground improvements at the Enlisted Men's Barracks, South Potomac Park, Washington.

From the opening of his private offices in 1887 to the time of his death, he acted as Engineer for the larger real estate operators of Washington, and in adjoining States, and had full charge of planning and developing many beautiful subdivisions in and around Washington.

On August 25, 1928, he was appointed by Governor Byrd, of Virginia, a member of the Commission to confer with a similar Commission of the State of Maryland for the purpose of settling a dispute which had arisen between the two States.

In 1910, he organized, with his son, Beaudric L. Howell, Assoc. M. Am. Soc. C. E., as Junior Member, the present firm of David J. Howell and Son, of Washington. He continued as an active member of this engineering firm until his death. Mr. Howell died suddenly of heart failure while engaged in an inspection of construction work on August 28, 1928.

Mr. Howell was a member of the New England Water Works Association, the American Water Works Association, and the Washington Society of Civil Engineers, and was a Certified Professional Engineer of the State of Virginia.

In 1887, he was married to Mary M. Gibson, who, with one son, survives him.

Mr. Howell was elected a Member of the American Society of Civil Engineers on September 5, 1900.

JULIUS MERRIAM HOWELLS, M. Am. Soc. C. E.*

DIED APRIL 22, 1927.

Julius Merriam Howells was born in Bowling Green, Ohio, on March 26, 1859, the son of Joseph and Ruth Merriam Howells. His childhood was spent on a farm in what he termed, "the Black Swamp", near Toledo, Ohio. His father, Joseph Howells, was progressive in thought, adopting the new school of medicine in his practice as a physician, and the new teachings of the Swedenborgian Church in his religion in place of the old orthodox beliefs. Remuneration from his medical practise among the farmers being inadequate for his large family, he moved to Richmond, Ind., where he entered the drug business. Joseph Howells' idea of justice was so unique that upon arriving at a degree of success, his conscience forbade him to enjoy such an easy life, so he again took up his more moderate practice as a physician. This quality of mind was inherited to some extent by his son, Julius, who loved his profession as an engineer, but had little regard for the money it brought him.

Julius Merriam Howells received his education at Earlham College, Indiana, and, later, from the late James E. Mills, M. Am. Soc. C. E., Geologist, of California. Like many others of the older engineers, Mr. Howells had his first practical experience in railroad location and construction for the Monarch Pass, Gunnison and Dolores Railroad, in Colorado (never built), and the Denver and Rio Grande Railroad in Marshall Pass and on the Rio Grande River, in Colorado. During 1881 and 1882 he was with the Maintenance of Way Department of the Pennsylvania Railroad Company, at Pittsburgh, Pa. From 1882 to 1884, he made studies of the tertiary history of the Sierra Nevadas, and mineral surveys under Mr. Mills, with Agassiz and Shaw, Geologists, his headquarters being at Quincy, Calif. From 1884 to 1888, he was Assistant City Engineer, and then City Engineer, of Richmond, Ind., at which time he became a Member of the Western Society of Engineers. From 1888 to 1892 he practised as Consulting Engineer in Chicago, Ill. His practice included work for the Municipal Investment Company, a Chicago firm, and he also served as Chief Engineer for Fostoria, Ohio, where he designed and built the entire system of water-works.

From 1892 to 1894, Mr. Howells was Chief Engineer of the Santa Fé, N. Mex., Water Company, at which time he did his first work in hydraulic-fill dam construction. He built the Santa Fé Canyon Dam, on which he experimented with the hydraulic-fill process. He then spent a year as Chief Engineer of the Tyler, Tex., Water Company, where he designed and constructed the Tyler Dam, the first hydraulic-fill dam ever built.† He also represented the Municipal Investment Company, of Chicago, as Engineer-Inspector of properties scattered over half the United States. In 1895, he became Chief Engineer of the Santa Fé Gas and Electric Company, for the Municipal Investment Company. From 1895 to 1898, he was Chief Engineer of the San Diego Flume

* Memoir prepared by Philip Schuyler, M. Am. Soc. C. E., and Hamilton R. Howells, Assoc. M. Am. Soc. C. E.

† "Reservoirs for Irrigation, Water Power, and Domestic Water Supply," by James Dix Schuyler, M. Am. Soc. C. E.

Company, and built the La Mesa Dam. In 1898, he designed the Pine Valley Dam, which was never completed. From 1899 to 1901 he was Consulting Engineer for the San Joaquin Electric Company, for which Company he built the Crane Valley Dam (hydraulic-fill and rock-fill).

In 1901 Mr. Howells became associated with the late James Dix Schuyler, M. Am. Soc. C. E., of Los Angeles, Calif., which association and friendship continued until the death of the latter. In 1903, he built the Lake Francis Dam (hydraulic-fill), for the Bay Counties Power Company, now the Pacific Gas and Electric Company. It was in 1901 that Mr. Howells conceived the Big Meadows and Feather River hydro-electric development; and from 1903 to 1905 he worked at interesting capital, promoting the Feather River project, obtaining early water rights, and commencing planning and construction.

From 1905 to 1909 he was employed by the Kobe Syndicate in the development of the Anglo-Japanese hydro-electric project, in Japan, following which he spent nearly a year in Switzerland. During 1910 to 1912 he was in Porto Rico as Chief Engineer of an irrigation system near Barceloneta, for which Mr. Schuyler was Consulting Engineer.

From 1912 to 1927 Mr. Howells carried on a general private consulting engineering practice in San Francisco, Calif., his clientele including the East Bay Water Company; the Western Canal Company; the Ocheco Irrigation District; the Reclamation District No. 784; the Montague and Granada Irrigation Districts; the Anglo and London Paris Bank; the Anglo California Trust Company; the James D. Phelan Estate; the Dodge Land Company; Girvin and Miller (bonds); Stephenson and Company (bonds); the Farm Land Investment Company of Fleishackers; the James Irvine Water Development, etc.

For this same period—1912 to 1927—he was Consulting Engineer for the Great Western Power Company for which he designed and built the Big Meadows Dam (hydraulic-fill) in 1913, and the Butte Valley Dam in 1923. In 1926, he engaged in the enlargement of the Big Meadows Dam.

Mr. Howells had an unusual imagination and the faculty of accurately visualizing structures in their completed state, long before their construction. His natural ability to estimate quantities and costs of engineering works amounted to genius. His associates would laboriously prepare an estimate only to find that they verified his simple estimate given weeks before without any apparent effort. He had a natural knowledge of proportion and logic, viewing all problems in their entirety without being confused by details. He was an idealist in his profession as he was in his private life. He had a rigid code of business ethics, and a very fair but strict idea of justice.

He readily grasped the economic and political aspects of problems and, due to this trait, found himself useful as an engineering diplomat and arbitrator in controversies among reclamation districts in co-operating with the State plan of reclamation. Loyalty to his friends and associates was probably his strongest trait. He took great pleasure in his work and keenly enjoyed the engineering profession. He was an exceedingly hard worker and preferred to work rather than to pursue the usual diversions of business men.

One of Mr. Howells striking characteristics was his unusual engineering economies. He constantly indicated to his associates where savings of money and effort could be made by using original and unique processes—many so simple that others overlooked them. To illustrate this, when the consolidation of an earth dam was being contemplated and intricate methods of compacting were being discussed, Mr. Howells accomplished a superior and cheap consolidation of the fill by arranging with a near-by goat farm to borrow a band of goats to drive to and fro across the fill. This ingenuity was also exhibited in his rather inventive turn of mind.

He was a member of the Institution of Civil Engineers of Great Britain and Chairman of the Economic Committee of the Immigration Section of the Commonwealth Club of California.

In his private life Mr. Howells was also the idealist. He stood for the very highest code of morals and might have given the impression of being intolerant of violators of his strict code. He was devoted to his home and family. He was a great lover of music and provided a musical training for each of his children. He was extremely interested in plant and animal life as well as in the geologic structure of the earth. He possessed a great familiarity with these subjects and made frequent allusions to them when in the field on business.

His other interests were: The Commonwealth Club and the League of Nations Non-Partisan Association. He stood for the Republican Party in politics. He was a member of the Swedenborgian Church, but during his last few years he was affiliated with the Unitarian Church, being interested in its educational lecture form of sermons.

He is survived by his widow, two sons, Hamilton R. Howells, Assoc. M. Am. Soc. C. E., and Merriam J. Howells, and two daughters, Mrs. George F. Buckingham and Mrs. Southall R. Pfund.

Mr. Howells was elected a Member of the American Society of Civil Engineers on June 4, 1902.

FRANK JAMES HUBBARD, M. Am. Soc. C. E.*

DIED FEBRUARY 23, 1927.

Frank James Hubbard was born on February 24, 1868, in Plainfield, N. J. He was the son of Joseph A. and Charlotte Elizabeth (Robertson) Hubbard. He was graduated from the Plainfield High School in the Class of 1887 and after his graduation spent nearly three years on a ranch in Wyoming for the benefit of his health.

From 1889 to 1893, Mr. Hubbard was employed in the office of the County Engineer of Union County, New Jersey, on highway construction, surveys for the mapping of Lapeer County, Michigan, and also in charge of street paving in Olean, N. Y. The following year he was a Special Student at the Massachusetts Institute of Technology, after which he entered the employ of the City of Plainfield as Assistant Engineer on the construction of its sewer system, serving in that capacity until 1896.

At the conclusion of his engagement with the City of Plainfield, Mr. Hubbard entered private practice, in which he was engaged until the time of his death. During this period he designed a sewer system for the Village of Sea Cliff, Long Island, served as Borough Engineer of North Plainfield, N. J., from 1897 to 1912 and from 1920 to 1926, and as County Engineer of Union County, from 1897 to 1911, designing and supervising the construction of bridges and paving. In 1903, he designed and installed a waterworks system for Alfred, N. Y., and, from 1915 to 1927, held the office of Borough Engineer of Dunellen, N. J. Just prior to his death he completed the work of designing a viaduct over a stream dividing the municipalities of Plainfield and North Plainfield.

Mr. Hubbard was a member of the Union County Chapter of the New Jersey Society of Professional Engineers, the Engineers' Club of Plainfield, and the Rotary Club of Plainfield, of which he was a Charter Member.

He served the community in which he lived in many positions of responsibility and trust and will ever be remembered for his interest and services in civic affairs. He had been a member of the Board of Education since 1915 and served as its President from 1923 until his death. During this period, he superintended the erection of a number of the finest school buildings in the city for which work he was eminently fitted by his engineering training and experience. In recognition of his work as a member of this Board, the City honored his memory by giving his name to its most recently constructed school building.

During his life Mr. Hubbard gave much time and attention to Muhlenberg Hospital; he was a member of its Board of Governors for nearly a quarter of a century and its President for four years. He also took an active interest in the building of the newer and larger fireproof structures of this institution, which construction he personally superintended.

* Memoir prepared by A. W. Vars, M. Am. Soc. C. E.

He was a member of the Seventh Day Baptist Church and was actively connected with several of its denominational interests. He was Treasurer of the Board of Trustees of the Seventh Day Baptist Memorial Fund which administers the endowment funds of that denomination.

In 1897, Mr. Hubbard was married to Bessie Evelyn Titsworth who survives him.

Mr. Hubbard was elected a Member of the American Society of Civil Engineers on April 12, 1926.

RICHARD LEWIS HUMPHREY, M. Am. Soc. C. E.*

DIED NOVEMBER 2, 1928.

Richard Lewis Humphrey was born in Marblehead, Mass., on October 19, 1869, the son of Richard Hales and Caroline (Curtis) Humphrey and a descendant of Benjamin Hales Humphreys, who came from Wales early in 1600 and settled in Marblehead. His father, who dropped the final "s" from his name, was a manufacturer of shoes in Marblehead and, later, in Philadelphia, Pa., and lived to be nearly ninety years of age. The son was graduated from the Central High School of Philadelphia with the degree of Bachelor of Arts in 1888 and Master of Arts in 1893, and from the University of Pennsylvania in 1891. He later received the degree of Civil Engineer from this institution, and subsequently took a special course in chemistry there.

Mr. Humphrey began his professional career in 1890, during the summer vacation of his Senior year, as Assistant and Topographer to the Division Engineer, on the Cumberland Gap Extension of the Louisville and Nashville Railroad Company. After his graduation, in 1891, he accepted a position as Draftsman in the service of the Philadelphia and Reading Railway Company, resigning in 1892 to become Draftsman and, later, Engineer in the Bureau of Surveys, Philadelphia. He was assigned, successively, to work on plans for the Cresheim Arch of the Wissahickon Creek high-level sewer, main and branch sewers, Front Street plate girder bridge (100-ft. span), and the drainage of the meadow lands in South Philadelphia. He was Designer and Engineer in charge of the Philadelphia Municipal Testing Laboratory. This laboratory grew in importance and in it much valuable research work was carried on, particularly in the development of the use of cement, concrete, and reinforced concrete as materials of construction, and Mr. Humphrey was considered a pioneer in work of this character.

In 1899, he became Engineer and General Manager of the Buckhorn Portland Cement Company, and during the ensuing four years he designed, built, and operated this Company's Portland cement plant at Manheim, W. Va. In 1903 he became a Consulting Engineer and Expert, serving the Pennsylvania Railroad Company, and other large corporations. He was Engineer in charge of the construction of the reinforced concrete Exhibition Building and of the assembling and erection of the collective American Portland cement exhibit and model testing laboratory of the American Portland Cement Manufacturers, which exhibit received the Grand Prize at the Louisiana Purchase Exposition, at St. Louis, Mo., in 1904. He served as Cement Expert of the United States Reclamation Service and Cement and Concrete Expert of the United States Geological Survey, and was Engineer in charge of the Government Structural Materials Testing Laboratories in St. Louis, Pittsburgh and Northampton, Pa., and Atlantic City, N. J., from 1905 to 1910, which laboratories he planned. He was Chairman of the Committee appointed by the Government to investigate the effects of alkali on concrete and rein-

* Memoir prepared by a Committee consisting of George S. Webster, Past-President, Am. Soc. C. E., and Samuel T. Wagner and S. M. Swaab, Members, Am. Soc. C. E.

forced concrete used in the construction of United States Reclamation Projects.

Mr. Humphrey also made, in 1906, an investigation for the Government of the effect of the San Francisco earthquake and fire on structures and structural materials, the results of which were published as a Government *Bulletin*. He established and designed the laboratory at Northampton, Pa., for the testing of the Portland cement used in the construction of the Panama Canal and was in charge of it until 1910. From 1910 until 1920 he was Consulting Engineer to the United States Bureau of Mines. During a part of 1910 he was also Consulting Engineer to the United States Bureau of Standards. He was the Expert retained in connection with the revision of the New York City Building Code in 1908 to 1909.

During the World War Mr. Humphrey was appointed (on October 7, 1917, as a "dollar a year" man) an Expert on the Raw Materials Committee, Council of National Defense, 1917-18. He was also appointed Director of Building Materials and of the Building Materials Division of the War Industries Board, which was created during the summer of 1918, and handled all matters relating to the conservation and distribution of building materials and supplies. He further served on the United States Highway Council, representing the War Industries Board in 1918. After the dissolution of the Board at the close of 1918 he continued to act for the Government in planning for the disposal of surplus building materials in such a way as to give the maximum price return to the Government and, at the same time, not unduly disturb the building materials market.

He was deeply interested in the welfare of the University of Pennsylvania. As President of the Engineering Alumni Society and as a Director of the General Alumni Society, he gave of his time and ability without stint. His duties were discharged with courage and fidelity, and he has left his impress on the affairs of this great educational institution.

Mr. Humphrey was an outstanding character. His activities were numerous, his energy great. He achieved much in the field of engineering, especially in such organizations as the American Society of Civil Engineers, the American Society for Testing Materials, and the American Concrete Institute.

In the American Society of Civil Engineers he was a leader in the movement to establish Local Sections and in having them take an important part in Society activities. The developments which followed created a new era in Society affairs and resulted in greater service being rendered to the membership throughout the country.

He was deeply interested in the registration of engineers and served as Chairman of a Committee of the Society to draft a uniform law on this subject to be adopted in all parts of the United States. Such a law having been enacted in Pennsylvania, Mr. Humphrey was appointed by the Governor of the State, as Chairman of the Board to administer the law, which position he held from the institution of this Board in 1921 until his death. He was largely instrumental in establishing the highest ideals for the Engineering Profession in the operations of the Board.

The high regard with which the members of the Board held Mr. Humphrey may be best expressed by the words which they addressed to him on December 19, 1924, as follows:

"For a leading and distinguished engineer to have made the sacrifice of time and energy in the service of the State, that you have made, affords a notable example to the members of the Engineering Profession of how one of their number can lay aside personal interests, and forego large private gain, to give to the State high service for the promotion of the public good and the elevation of the profession of engineering."

In January, 1928, Mr. Humphrey was appointed a member of the Special Committee of the Board of Direction of the Society on Technical Expansion. On this Committee, as well as on all committees on which he served, he always gave unselfishly of his best thought in advancing the interests of the Society.

In the American Society for Testing Materials and in the American Concrete Institute, he had a most important part in the development of cement and concrete specifications and was recognized as an authority on these subjects.

Mr. Humphrey received a Gold Medal as collaborator at the Louisiana Purchase Exposition, St. Louis, Mo., in 1904, and the Medal of the British Concrete Institute in 1911. He was United States Delegate and Honorary President of the International Congress for Testing Materials at Copenhagen, Denmark, in 1909. He was appointed by President Roosevelt as a Member of the National Advisory Board on Fuel and Structural Materials and was its Secretary from 1905 to 1910.

He was a member of the Institution of Civil Engineers (Great Britain); Société des Ingénieurs Civils de France (Vice-President, American Section); British Fire Prevention Committee; National Fire Prevention Association; a Fellow of the American Association for the Advancement of Science; a member of the National Geographic Society; American Chemical Society; Society of Chemical Industry (British); Société Industrielle Chimie; International Association for Testing Materials (Chairman of the International Committee on Constancy of Volume of Cement); American Society for Testing Materials (organized it and was its first Secretary, 1898-1900); American Railway Engineering Association; National Association of Cement Users (First President, 1905-13); American Concrete Institute (First President, 1913-15; First Honorary Member, 1926); awarded the Wason Medal in 1925 for two papers, "Twenty Years of Concrete" and "The Promise of Future Development"; Oesterreichischer Béton Verein; American Concrete Pipe Association; Secretary of the Joint Committee on Concrete and Reinforced Concrete, the constituent organizations of which were the National Societies of Civil Engineers, Testing Materials, American Railway Engineering Association, the American Concrete Institute, and the Portland Cement Association; Chairman of the Joint Committee on Specifications for Concrete and Reinforced Concrete (since 1920); Secretary of the Committee on Uniform Tests for Cement; Secretary of the Committee on Concrete of the American Society for Testing Materials; and a member of the Executive Committee of the Joint Conference Committee on Uniform Methods of Tests and Standard Specifications for Cement, consisting

of representatives of the American Society of Civil Engineers, American Society for Testing Materials, and the United States Government. He was also Chairman of the governing body of American Engineering Council and Chairman of the Committee on Procedure, representing those National Engineering Societies which created the Federated American Engineering Society.

Mr. Humphrey was a man of remarkable ability and of comprehensive good judgment and rendered many useful services in the engineering world. He was especially helpful in his contacts with young men entering the profession and was active in organizing Student Chapters in the Society.

He was an indefatigable worker, thorough and painstaking in every thing he undertook, always an earnest and forceful advocate of the measures which he believed to be for the best interest of the Engineering Profession.

He was a staunch friend, a charming and lovable companion, and it was a privilege and an inspiration to be associated with him. The loss of his influence and counsel will be keenly felt at future meetings of the technical societies of which he was a member.

He is survived by his widow, Anna Kay Humphrey, and two sons.

Mr. Humphrey was elected an Associate Member of the American Society of Civil Engineers on May 5, 1897, and a Member on May 3, 1904. He served as Director from 1921 to 1923 and as Vice-President in 1925 and 1926.

FRED ATWOOD JONES, M. Am. Soc. C. E.*

DIED MAY 22, 1928.

Fred Atwood Jones was born in Dallas, Tex., on August 23, 1875, the son of Frank Jones and Nannie E. (Hunt) Jones, natives of Missouri, his father having gone to Texas as a lumberman with the William Cameron Interests.

When the boy was three years of age the family moved to Bonham, Tex., where he was educated, having been graduated from Fannin College, at the age of fifteen. He then attended and was graduated from Richmond College, at Richmond, Va., receiving his Bachelor of Arts Degree from that institution in 1894. He then enrolled in Cornell University, at Ithaca, N. Y., to take professional courses and, in 1898, he received the degree of Electrical and Mechanical Engineer.

After his graduation from Cornell, Mr. Jones began the practice of miscellaneous engineering and surveying in Northern Texas. Later, he took the Student's Course with the General Electric Company, at Schenectady, N. Y., and for two years he was engaged in the Testing and Railway Engineering Departments of that Company.

In January, 1902, Mr. Jones opened an engineering office in Houston, Tex. He served as Consulting Engineer during the building of the power station of the Southern Pacific Terminal Company, at Galveston, Tex., as well as that of the Corsicana Gas and Electric Company, and many other such plants, at the same time submitting a great number of engineering reports. By 1908, he had gathered about him other highly efficient engineers and builders. To list his chief works would be to review the great building operations of many large centers of Texas, from the older cities to the new-born oil centers. Among other structures at Dallas that may be credited to his organization, are the Dallas Country Club; the Sumpter (now the Central Bank) Building; the Southern Methodist University Administration Building; the City Hall, which is a lasting monument of beauty and stability, comparing well with the foremost municipal buildings of the largest cities; the Southland Life Building; the eight-story addition to the Interurban Building; the White Rock Reservoir; the Dallas Filtration Plant; the Texas "Farm & Ranch" Building; the Palace Theatre, which ranks with the leading theatre buildings of America; and the interurban lines leading from Dallas to Sherman, from Dallas to Corsicana, and from Dallas to Waco.

At Houston, the Rossonian Apartments, the Sunset Hospital, the Bender Hotel, and the Levy Brothers Department Store are some of his work in that city, as well as the Telephone Exchange, which is also to his credit. There is also the American National Insurance Building, at Galveston, Tex.; the Nueces Hotel, at Corpus Christi, Tex.; the Paso Del Norte Hotel, at El Paso, Tex.; Trinity University Dormitory, at Waxahachie, Tex.; the National Supply Building and the Republic Supply Building, at Ranger, Tex.; the Atlas Supply Company Building, at Eastland, Tex.; and the County Hospital, the Moulton Hotel, and the American Trust and Savings Bank Building, a twenty-story structure, all at Birmingham, Ala.

* Memoir prepared from information on file at the Headquarters of the Society.

In addition, Mr. Jones prepared the cost review report on the Roosevelt Dam, Salt River Valley Project, which brought into usefulness land valued at $10 000 000. He also made the survey and report for the proposed Dallas-Wichita Falls Interurban Line, and his organization handled the construction of Camp McArthur Cantonment, Rich Field (an aviation camp), and a large hospital for the Government, at Waco.

During the World War, he was Chairman of the Military Training Camps Association, and conducted examinations of civilians for Officers' Training Camps. He also represented the Secretary of War in visiting training camps and aviation fields in Texas and conferring with officers on various matters.

On September 5, 1910, Mr. Jones was married to Gussie Holland, the daughter of the late Col. Frank P. Holland, a former Mayor of Dallas, and the owner of *Farm & Ranch*, and *Holland's Magazine*. They had three children, Fred A. Jr., Robert Holland, and Latham Fletcher Jones. He is also survived by his brothers, the Hon. Frank C. Jones, of Houston, and C. A. Jones, of Greenville, Tex.

Although quiet and unassuming, Mr. Jones was one of the most prominent engineers and builders of the Southwest. He allied himself with the social and civic life as well as with the commercial affairs in each city in which he resided. He was a Shriner, a member of Hella Temple, Dallas, and a member of the City Club, the Dallas Country Club, and the Brook Hollow Golf Club. He was one of the oldest members of the American Institute of Electrical Engineers.

Mr. Jones was elected a Member of the American Society of Civil Engineers on December 6, 1915.

HAROLD VINCENT JOSLIN, M. Am. Soc. C. E.*

DIED NOVEMBER 3, 1928.

Harold Vincent Joslin, the son of William C. and Elizabeth Florine (Freeman) Joslin, was born on June 22, 1883, at Belleville, N. Y. He was educated at Clinton Liberal Institute, Fort Plain, N. Y., and at Brown University, Providence, R. I., having been graduated with the degree of Bachelor of Arts in 1904.

After graduation, Mr. Joslin's first work was with the White Mountain Lumber Company, of Conway, N. H., where he was engaged in land surveys and in the location of narrow-gauge railways. From there, he went to Newark, N. J., where he served for a short time as Draftsman with the New York and New Jersey Telephone Company. He then became associated with Betts Academy, at Stamford, Conn., as Instructor in Mathematics.

In October, 1905, he entered the service of the Norfolk-Southern Railway, in Norfolk, Va., where he won rapid promotion. He served, successively, in the positions of Draftsman, Levelman, Transitman, Resident Engineer, Assistant Engineer of Maintenance of Way, and First Assistant to the Chief Engineer, until March, 1910. He then accepted a position with the City of Norfolk, and was in charge of surveys for water supply until June, 1911.

From June, 1911, until March, 1912, Mr. Joslin served as Construction Engineer for the Yadkin River Power Company, a subsidiary of the Carolina Power and Light Company, of Raleigh, N. C., on the construction of a hydro-electric plant at Pee Dee, N. C.; this work included the design and layout of the plant. He was then engaged by the Norfolk-Southern Railway Company for a few months as Locating Engineer on its Raleigh-Charlotte Branch. At this time he re-entered the service of the Yadkin River Power Company and the Carolina Power and Light Company, as Assistant to the Construction Manager and Cost Engineer and served in this capacity until April, 1913, when he went to Grace, Idaho, as Assistant to the Construction Superintendent and Cost Engineer of the Phœnix Construction Company. In this capacity he was engaged on hydro-electric surveys, design and installation of plant and subsidiary structures, and general construction.

In August, 1914, he returned to the Carolina Power and Light Company and Allied Companies, at Raleigh, where he had charge of the design and construction of substations, transmission lines, and miscellaneous work of a similar nature, until May, 1918, when he was appointed General Field Cost Engineer for the United States Housing Corporation, at Washington, D. C. He served one year in this capacity and another year as Manager of the Wilson Housing Corporation, of Wilson, N. C.

Mr. Joslin then returned to Raleigh and became associated with the North Carolina State Highway Commission as Assistant to the Chairman. He served this Commission until October, 1923, when he resigned to return to the Carolina Power and Light Company, for which he handled the design of

* Memoir prepared by George F. Syme, M. Am. Soc. C. E.

transmission lines, bridges, buildings, and other structures, and also supervised their location and construction, as well as the surveys and investigations for water-power development. He held this position at the time of his death, on November 3, 1928.

Mr. Joslin was a faithful communicant of the Protestant Episcopal Church, a member of Phi Beta Kappa Society, Delta Kappa Epsilon Fraternity, and the William G. Hill Masonic Lodge No. 218, of Raleigh.

He was a member of the North Carolina Society of Engineers and of the Raleigh Engineers' Club, having served the latter as Vice-President. He took a very active part and a deep interest in all matters pertaining to these local engineering societies, and whenever possible attended all their meetings.

He ably served on many engineering committees appointed to co-operate with various civic bodies in the handling of local public problems. His advice was eagerly and frequently sought by his contemporaries, who appreciated and respected his sound judgment on technical matters, and the ever-present friendliness with which he responded.

Mr. Joslin was well known professionally and socially throughout his adopted State, and his memory will live in the hearts of his friends, until they, too, answer the last call. After that, the enduring works he left behind will direct future generations to the accomplishments of an engineer, which time cannot erase. "By their works shall ye know them."

He was an outstanding figure in his community, and his lovable disposition, courtly manners, and splendid character endeared him to all who were privileged to know him. His untimely passing spread universal sorrow among his friends and is a distinct loss to his employers and to his profession.

Mr. Joslin was married on December 1, 1908, to Annie Devereux Hinsdale, of Raleigh, who, with four children, Ellen Devereux, Devereux, William, and John Hinsdale, survives him.

Mr. Joslin was elected a Junior of the American Society of Civil Engineers on April 6, 1909; an Associate Member on February 6, 1912; and a Member on October 8, 1918.

HOWARD GEORGE KELLEY, M. Am. Soc. C. E.*

DIED MAY 15, 1928.

Howard George Kelley was born in Philadelphia, Pa., on January 12, 1858. He was educated in the public schools of Philadelphia and was a student in civil engineering at the Polytechnic College of Pennsylvania.

On leaving school Mr. Kelley desired to follow the lure of the sea which inclination was derived from the experiences of his grandfather and his father, both sea-faring men, although his father in middle age had given up his ship and become a merchant in Philadelphia. Mr. Kelley, however, engaged to sail on a small ship of less than 400 tons from San Francisco, Calif. He became well versed in the study of navigation which he found a pleasing diversion. His grandmother, who had taken care of him in his youth, prevailed upon him to give up his sea-faring proclivities and he became a Land Surveyor in California. His later work, fortunately, gave him numerous opportunities to indulge in his love of and pleasure in ocean travel. He crossed the Atlantic a dozen times in the line of duty and during the winter of 1924 and 1925 he and Mrs. Kelley enjoyed a trip around the world.

The record of his services is most interesting in its diversity and its steady accumulations of success. It demonstrates his close attention to his chosen profession and shows the result of the intensity with which he carried on every duty in the line of his work.

In April, 1881, Mr. Kelley entered the service of the Northern Pacific Railway Company at Tacoma, Washington Territory, on the original preliminary and location surveys from Kalama, Washington Territory, to Portland, Ore., running lines on both sides of the Columbia and Willamette Rivers. He then joined the crew on the preliminary and location surveys from Sand Point, Idaho, on Lake Pend'Oreille, around the north and east sides of the lake, and thence up the Valley of Clarks Fork of the Columbia River, through the Cœur d'Alene and Cabinet Mountains, to its junction with the Missoula River in Montana, to the point now known as Paradise, at that time a totally uninhabited country.

On the completion of the surveys, he was appointed Transitman for the Division Engineer in charge of bridges and, on the retirement of the Division Engineer on account of illness, Mr. Kelley was placed in charge of the party and completed the work to the Missoula River, including the bridge over Clarks Fork at that point, then known as the "Third Crossing". This work included the design and construction of bridges, trestles, viaducts, piers, and abutments, all built of timber, and performed by company forces. In July, 1883, he was transferred to the Kalama-Portland Division as Resident Engineer in charge of bridge construction. Later in the autumn, he was put in charge of the work on the Upper Yakima River for about two months.

Mr. Kelley was then engaged for several months in making a survey of the water-front at Seattle, Wash., on which the later improvements by the Northern Pacific Railway Company were based. Subsequently, he was placed in charge

* Memoir prepared by Carl H. Gerber, Esq., Omaha, Nebr.

of the field work on the survey and soundings of Tacoma Harbor and the Lower Puyallup River. He was engaged on reconnaissance in the Cœur d'Alene Mountains, in Idaho; on the western slopes of the Cascade Range, in Washington Territory; and in the Olympic Mountains south of the Straits of San Juan de Fuca, also in Washington Territory. His work with the Northern Pacific Railway Company and allied service covered a period of four years, terminating in the spring of 1885.

During the remainder of 1885, and for the succeeding year, Mr. Kelley was engaged in mining, his work including nearly every job pertaining to a mine, incidentally, also, learning the blacksmith trade, which resulted in his being placed in charge of the development of the Mapleton and Champion Mines of the Summit Mountain Mining District.

In December, 1886, he was appointed Resident Engineer of Maintenance of Way on the St. Louis, Arkansas and Texas Railway, and, later, on the St. Louis Southwestern Railway, which extended from Birds Point, Mo., opposite Cairo, Ill., southward through Missouri, Arkansas and Texas, 395 miles of this railway having been changed from narrow to standard gauge at that time. The position of Superintendent of Bridges and Buildings was later added to his duties as Resident Engineer.

The entire original railway was rebuilt, including all the bridges over the rivers which constituted the principal drainage lines of that section of the country; one item was the sinking of two pneumatic piers with a penetration of 88 ft. below the surface of the water. This work was started under contract, but completed by company forces under Mr. Kelley's direction. During this period the railway was being extended in Missouri, Arkansas, Louisiana, and Texas, to a total of 1 228 miles, and on the completion of the grading and track-laying of these extensions, he took charge of the construction of the permanent structures and was appointed Chief Engineer.

As Chief Engineer Mr. Kelley continued in direct charge of bridges and buildings; he located and built the line from Delta to Grays Point, Mo., together with the yard and incline at Grays Point, and also located the first line to the Mississippi River for a bridge to Thebes, Ill.; he made the reconnaissance personally and reported on the terminal property required for the route to Dallas on which the line was built; he also made several reconnaissances in Texas and reports on other smaller properties.

The valuation of the St. Louis Southwestern Railway Company's properties in Texas, aggregating between 500 and 600 miles, made under a General Order of the State Railroad Commission, was carried out under his personal direction and supervision. His services with this Company extended throughout a period of eleven years.

On March 1, 1897, Mr. Kelley resigned his position to become Chief Engineer of the Minneapolis and St. Louis Railroad Company, being retained for another year as Consulting Engineer for the St. Louis Southwestern Railway Company. As Chief Engineer of the Minneapolis and St. Louis he had charge of maintenance, including bridges and buildings, real estate, and new construction.

While in the service of the Minneapolis and St. Louis Railroad Company, Mr. Kelley made an examination and report of the Iowa Central Railway, about 500 miles, and of the Des Moines and Fort Dodge Railway, about 139 miles, and on the basis of his report these lines were acquired and became part of the Minneapolis and St. Louis System. A number of important changes and extensions were made, including reconstruction for improvement in grades and alignment on the main line of this System, as well as on the Iowa Central Railway, all of which was heavy work, covering a distance of 296 miles. This reconstruction was performed by company forces with the exception of 15 miles on the Minneapolis and St. Louis Railroad between Hopkins Junction and Chaska, Minn. and 7 miles on the Iowa Central Railway which was done under contract. In addition, there was also the location and construction of 363 miles of new lines for the Minneapolis and St. Louis Railroad Company in Minnesota, Iowa, and South Dakota, and a number of reconnaissance examinations for other projected lines. The valuation of these lines, made by order of the Minnesota Railroad and Warehouse Commission, was under Mr. Kelley's direct supervision. Throughout the work he had charge of the operation of the lines under construction. His services with the Minneapolis and St. Louis Railway Company continued for a period of nine years.

In July, 1907, Mr. Kelley was appointed Chief Engineer of the Grand Trunk Railway System, for both the American and Canadian lines, aggregating 4 785 miles. After four years in that position, in October, 1911, he was made Vice-President in charge of operation, maintenance, and construction. Six years later, in October, 1917, he was elected President of the entire Grand Trunk System, the Grand Trunk Pacific Railway, and its steamship lines, in all more than 8 000 miles of railway. He also served as Chairman of the Board of the Central Vermont Railway Company, President of the Toronto Terminal Company, and Director of the Chicago and Western Indiana Railroad Company, and the Belt Railway Company of Chicago, which positions he retained until the Grand Trunk Railway System was taken over and made part of the Dominion of Canada Government Railways in 1922.

Important among the noteworthy items accomplished during this period of fifteen years was a substantial improvement of the physical condition of the property and a very heavy increase in business; also the reconstruction of bridges over the St. Lawrence and Niagara Rivers; the improvement of grade line and elimination of grade crossings from York through Toronto to Mimico, Ont., Canada; the construction of the Union Station building, at Toronto; the elimination of grade crossings, covering several miles, at Detroit, Mich.; the preliminary plans for the abolition of grade crossings in the City of Montreal, Que., Canada, covering more than six miles and including three terminal yards, and the preliminary plans for a new terminal station; the plans and estimates for an entirely new terminal for the Chicago and Western Indiana Railroad Company in Chicago, Ill.; the electrification of the St. Clair River Tunnel from Sarnia, Ont., Canada, to Port Huron, Mich.; the construction of the Montreal and Southern Counties Railway, an electric freight and passenger line of 44 miles; the construction of 4.5 miles of new line

between Birch and Wybridge, Ont., Canada, and 18.8 miles of new line from Cass City to Bad Axe, Mich.

The valuation of the American lines under orders from the Interstate Commerce Commission, including the lines of the Central Vermont Railroad Company, was started and subsequently carried on by the Valuation Department of the Grand Trunk Railway System. The valuation of the entire Grand Trunk System, together with a report on its physical condition, was made in 1920 and 1921, in connection with the arbitration proceedings under which the property was taken by the Dominion Government. This included all the owned and partly owned properties of the System; the Grand Trunk Railway, American and Canadian Lines; the Grand Trunk Pacific Railroad and Steamship Lines; the Central Vermont Railroad; the Detroit Terminal Company; the Toledo Terminal Company; the Detroit and Toledo Shore Line Railroad; the Chicago and Western Indiana Railroad and Belt Railway of Chicago; the Montreal and Southern Counties Railroad (Electric); the Oshawa Railway (Electric); the car ferries on Lakes Michigan and Ontario; the Central Vermont Transportation Company steamships; the Grain Elevator Companies; the Hotel Companies, and a number of other properties.

A noteworthy accomplishment, demonstrating Mr. Kelley's ability, energy, and tireless attention to meet emergencies, was his success in directing the movement of men, munitions, and supplies, occasioned by the World War. The record shows efficient and prompt movement of all traffic and, further, that no ship assigned to transportation overseas for Canada's participation in the war was delayed more than 24 hours in the whole 4-year period.

At the close of his services with the Grand Trunk System, he had planned to take a well-earned vacation and intended to go on a purely pleasure trip to Europe, but before he could put that plan into effect he was called to Boston, Mass., for a conference with the members of the Joint New England Railroad Committee, and in October, 1922, was selected as Technical Adviser to that Committee which he served until November, 1923, when the work was completed.

The Joint New England Railroad Committee was composed of the Chairmen of six State Committees appointed in the six New England States by their respective Governors. The members had been selected from among the leading bankers and business men for the purpose of studying the transportation and industrial situation and to report on such study with recommendations for the future advancement of New England and for the consolidation of the railways.

The work required a study of the manufacturing industrial situation of New England, the present and possible future markets, the transportation facilities to reach such markets, the various routes of distribution by rail, ocean, and lake, and the possible development of the principal ocean ports. It also included an inspection, study, and analysis of each of the several railroads of New England, their financial structures, operating conditions, methods and results, maintenance, facilities, requirements, and the probable effect on them and on the New England industries of the several methods of possible consolidation.

It involved a series of public hearings held in Boston, Hartford, Conn., Providence, R. I., and Montpelier, Vt., the States of New Hampshire and Maine being heard in Boston. The published report of the Committee required a book of 334 pages, supplemented by maps and charts, but the actual data assembled filled several thousand pages.*

From June until late in November, 1924, Mr. Kelley was engaged upon an analysis and report covering the location, construction, operation, value, and earning power of the Virginian Railway Company, together with its branches and terminal properties. This study included a complete review of this property from its inception to date (1902 to 1924) and covered the original location, operating grades secured, character of country traversed, type and character of construction, conditions under which construction had been conducted, the motive power and equipment, the operating results throughout the period of its existence, the business, past, present, and prospective, available for traffic and, finally, the earning power and value of the railway.

This was supplemented during the months of October and November, 1925, by a re-arrangement of the data contained in the report of 1924 for direct testimony presented in hearing before the Bureau of Valuation of the Interstate Commerce Commission in connection with the valuation case of the Virginian Railway Company.

In November, 1925, within a week after the completion of the Virginian hearing, Mr. Kelley left New York City to go to Russia to make an examination of two railways in Southern Russia, aggregating more than 3 000 miles.

The work involved a detailed inspection, study and analysis of the roadway, engine terminals, shop facilities, station and other buildings, and all other physical elements; an analysis of operating performances and statistics covering the period of years after the World War and for several years prior to the war; an estimate for the completion of the rehabilitation work undertaken by the Russian Government and the requirements for increasing the capacity and extending the lines of these two railways to properly serve the coal, iron ore, and manganese mines and the several steel plants in the Ukraine Province. It also included the examination of the seaports on the Black Sea. Nearly three months were required to complete the inspection and to obtain the desired data in Russia. A report on the project was compiled after Mr. Kelley's return to New York City and completed in June, 1926.

During August and September, 1926, an analysis and report were made of the Buffalo, Rochester and Pittsburgh Railroad Company, which report was used in the testimony presented before the Interstate Commerce Commission in connection with the hearing relating to the proposed lease of the Buffalo, Rochester and Pittsburgh by the Delaware and Hudson Company to form a part of the Loree project for an additional trunk line from New York City westward.

On May 20, 1927, Mr. Kelley again sailed from New York City, this time to make a study and an analysis of the State Railways of the Kingdom of Roumania, including their location, physical condition, equipment, operating

* Part of Interstate Commerce Comm. Docket, 12964.

methods and results, and earning power. These lines covered a total of 11 085 km., or 6 888 miles, including 150 miles of second track. The work involved a detailed inspection of the physical properties, the obtaining of data on operating performances and results, and an estimate for the rehabilitation and extensions to serve properly the traffic requirements of Roumania, including all commodities produced or transported within and throughout that country.

Mr. Kelley returned to New York on August 26, 1927, and for the next three months was occupied in compiling a report on his findings which he completed in December, 1927, at Omaha, Nebr. Subsequently, he was called to New York (January, 1928) for discussion and conferences in connection with both the Russian and Roumanian work. Concluding his stay in New York, he went to his home in Dallas, Tex., to attend to an accumulation of matters relating to his personal affairs and in April, 1928, went to San Diego, Calif., for rest and recreation.

His plans at that time were made in expectation of further service in connection with the two foreign projects, and he anticipated returning to Europe some time during June or July, 1928, to continue the investigations and conferences incident to the negotiations with the two Governments; but fate willed otherwise, and he passed on to that bourne from which no man returns on May 15, 1928, at San Diego.

Mr. Kelley was affiliated with numerous religious, fraternal, social, and business organizations, including the Protestant Episcopal Church and the Church of England; the Masonic Order; the Mount Royal Club; the Forest and Stream and Mt. Bruno Country Clubs (Montreal); the New York Yacht Club (New York City), the Congressional Country Club (Washington, D. C.), and many others.

He was a member of the Executive Committee of the American Railway Association from June, 1919, to January, 1923; the Standing Committee of the Association of Railway Executives from its inception until he retired from the Grand Trunk System; the Committee on Relations (now Section V) of the American Railway Association from 1911 to 1921, inclusive; the Per Diem Rules Arbitration Committee (one of five members); the American Railway Engineering Association, of which he was President for two years; the Institution of Civil Engineers, of London, England; the Canadian Institute of Civil Engineers; and a Director of the Royal Trust Company of Canada, from 1917 until his death.

Mr. Kelley was married in 1897 to Cora Lingo, of Denison, Tex., who survives him. He is also survived by two brothers, Walter S. Kelley, of Wheeling, W. Va., and Henry Kelley, of Big Spring, Tex.; and one sister, Mrs. Edith Homan, of Oklahoma City, Okla.

The passing of Howard G. Kelley is a grievous loss and he is mourned by many. His character and personality were of the highest type, his ability recognized by all, and his energy and thoroughness known to those with whom he came in contact. He was honored and respected as was his due.

Kindly and considerate, appreciative and generous, with that unusual understanding of conditions and personal temperament affecting those who worked

with him and for him; always ready to listen and to give sound counsel to every one; charitable to all; and the most delightful companion in every phase of life—these are only a few of the excellent attributes which he possessed.

His power of analysis and his sound judgment were accepted in determining methods and rules for guidance in the successful direction and advancement of transportation affairs by the owners and executives of the railways of the United States and Canada. His capacity for work seemed almost unlimited and his success in the railroad and business world are proof of his motto, "Without work, nothing".

Mr. Kelley was elected a Member of the American Society of Civil Engineers on May 1, 1889.

JAMES HENRY KENNEDY, M. Am. Soc. C. E.*

DIED OCTOBER 21, 1927.

The passing of James Henry Kennedy laid to rest one more of those fine old personalities who link this somewhat materialistic age with the romance of the railway builders of 40 or 50 years ago. That splendid group of pioneers have given much color to the history of National development on this Continent. Their part in those great enterprises which traversed the vast and beautiful expanse of Nature in the Northern and Western wilds, fashioned in them a nobility of character and purpose which has endeared them to all and rightly added to the pride and pleasure in their achievements. As Mr. Kennedy was an honored and respected representative of that passing generation of nation builders, it seems fitting that this brief narrative of his life should endeavor to retain at least some trace of the spirit which lifted him out of the humdrum of uneventful existence into adventure and success as a pioneer engineer in a realm of great and noble enterprise.

James Henry Kennedy was born on March 3, 1852, at Stittsville, Ont., Canada, which is just a few miles from Ottawa. His boyhood was spent on a farm in Elgin County, Ontario, near St. Thomas. There he developed a sturdy physique, a clean healthy mind, and a stirring ambition. His religious ideals were fostered in a Baptist atmosphere, and the habits of youth remained constantly with him to the end.

His early education was received at a rural school, and it was not until his twenty-fifth year that he commenced his higher education at Woodstock College, Ontario. Two years later, in October, 1879, he entered, as a Freshman, the old School of Practical Science, now recognized as the Faculty of Applied Science and Engineering of the University of Toronto. Under the personal instruction of the late Dean John Galbraith, with whom he formed a lasting friendship, he was graduated with honors in Civil Engineering in 1882, at the age of thirty. This marked the commencement of his professional career.

It was typical of Mr. Kennedy's modesty that he habitually termed himself "an awful duffer at school", but the fact that he attempted and gained a university education without a single failure, at such a mature age, is significant of his strength of character and mental ability. During his latter days he was the recipient of much honor and frequent congratulations in recognition of his being the oldest living graduate of the School of Practical Science, and the possessor of the degree of Civil Engineer awarded to him by the University of Toronto in 1886.

His first engineering job was as Rodman and Field Draftsman on a survey for the Canadian Pacific Railway during the summer of 1882, but in September of that year he was appointed Assistant Engineer on the re-measurement of Section "B", and in April, 1883, was promoted to be Assistant Engineer on the location and construction of the Lake Superior Division, where he continued until March, 1885.

* Memoir prepared by P. H. Buchan, Esq., Vancouver, B. C., Canada.

After spending three years in the wilds, Mr. Kennedy evidently felt the attraction of civilization for, in July, 1885, he was articled to A. D. Baikie, Provincial Land Surveyor, at St. Thomas, with whom he remained until he obtained his Ontario Land Surveyor's certificate (O. L. S.) in the spring of 1887. In May, 1887, he was re-engaged by the Canadian Pacific Railway Company as Locating Engineer on the Detroit Extension, between Woodstock and London, Ont., and afterward on the location and construction of the Temiscouata Railway in Quebec and New Brunswick. In May, 1889, he went to Minnesota to enter private practice in Winona as Deputy County Surveyor with the late M. S. Parker, M. Am. Soc. C. E.; but shortly afterward he was employed as Inspector of Tracklaying on the Neighart Branch of the Montana Central Railway.

For the next five years, Mr. Kennedy was very active on railway construction in the United States. In July, 1889, his services were engaged by the Great Northern Railway Company as Engineer in charge of surveys and construction on the Rocky Mountain Division of the Pacific Extension in Montana, through what is now known as Glacier National Park. To have progressed thus far, from an inexperienced Rodman on a survey party, in only seven years, points strongly to his enthusiasm and skill. He remained with this great project until March, 1892, after which he spent eighteen months on the construction of the Minneapolis, St. Paul, and Sault Ste. Marie Railway extensions in North Dakota. During this period he made examinations and reports on all the bridges in the "Soo" System for the late W. W. Rich, M. Am. Soc. C. E., then Chief Engineer of the "Soo" Lines.

In August, 1894, the spell was broken for the second time, and Mr. Kennedy resumed private practice as a Land Surveyor under his own name at St. Thomas. Concurrently, he became interested in a machine shop and bicycle factory in that city. However, three years of life in civilized parts appeared to satisfy him, for in February, 1898, he engaged with MacKenzie and Mann, under the direction of Mr. T. H. White, to go to the Stickine River in charge of Yukon Railway surveys between Telegraph Creek and Teslin Lake, in British Columbia. He remained there until mid-summer and then went south to take charge of the location of a line in the Okanagan, from Penticton to Midway, B. C. Later, he made several surveys for the British American Coal Company in the vicinity of Crows' Nest Pass, which occupied him until the spring of 1900.

He spent two years in British Columbia, his future home, and then returned to Ontario, where he was engaged to locate an electric railway from London to Port Stanley, near the scenes of his boyhood. The call of the wilderness, however, soon lured him away to Michipicoton Harbor, where he took charge of three survey parties for the Algoma Central Railway Company, from the great Algoma iron mines to Sault Ste. Marie. It was during the winter of 1900-01 that he made a reconnaissance on snowshoes down the Agawa River to Agawa Bay, in company with the late William McCarthy.

Having reached the age of forty-nine years, Mr. Kennedy once more turned his face westward, this time to stay. In February, 1901, he re-entered

the service of MacKenzie and Mann as Assistant Chief Engineer on the construction of the Vancouver, Victoria and Eastern Railway and Navigation Company's line from Penticton to Midway, on the location surveyed by himself in 1899. This line was acquired by the Great Northern Railway Company during 1901, and Mr. Kennedy acted in the same capacity with the latter Company. Nine years had elapsed since his previous engagement with the Great Northern in Glacier National Park. It was not long before he was promoted to the post of Chief Engineer of the Vancouver, Victoria, and Eastern Railway Company, from Laurier to the Pacific Coast, which position he retained until his retirement on August 15, 1915, at the age of sixty-three.

Much of his work in this connection involved heavy construction through rough mountainous country, the most difficult portions being the branch line from Laurier to Grand Forks, B. C., through the Kettle Valley, and the longer section of main line between Chopaka and Brookmere through the Similkameen and Tulameen Valleys. This construction called for a high degree of engineering initiative and skill to bring it to a successful conclusion, and Mr. Kennedy believed it to be the most difficult piece of railway construction he had ever undertaken. His retirement completed seventeen years with the Great Northern Railway Company on railway extensions.

In 1916, he opened an office for consulting practice at Vancouver, specializing in railway construction. He was also engaged on inspection work for the British Columbian Government during 1916 and 1917, on the construction of the Pacific Great Eastern Railway. From 1918 to 1922 he was retained by the Brooks, Scanlon, O'Brien Logging Company as Consulting Engineer on logging railway construction. In the summer of 1923 he made an inspection and report on the Edmonton, Dunvegan, and British Columbia Railway for the Alberta Government, and did further work of the same nature on this line in the late fall of 1926. During 1924 and 1925 he acted as Inspector of State Highway work for the Great Northern Railway Company at Rock Island, Wash.

That Mr. Kennedy kept in harness until he was nearly seventy-five years of age is no slight tribute to his splendid physical and mental qualities. His professional career covered a period of forty-five years. On January 20, 1887, he joined the Canadian Society of Civil Engineers as an Associate Member, his election to Membership occurring on May 18, 1893. Toward the close of his career he was granted Life Membership by the Engineering Institute of Canada (the successor of the Canadian Society of Civil Engineers) and also by the Society.

On October 6, 1923, Mr. Kennedy sat for a group photograph comprising T. H. White and the late H. J. Cambie, Members of the Engineering Institute of Canada, and himself, in commemoration of their distinguished services as Pioneer Railway Builders in British Columbia. An enlargement of this group was presented by the Institute to the Provincial Library at Victoria in 1924. This group is unique for several reasons, not the least interesting being the fact that Mr. Kennedy, although seventy-two, was the youngest member of the trio.

Among his varied interests during his later years, he was Vice-President of the Pacific Investment Corporation of Vancouver, and served as a member of the Executive of the Convocation of the University of British Columbia. He was a member and regular attendant of the First Baptist Church of Vancouver.

On June 30, 1884, he was married to Annie Huntley, at Toronto, Ont. He is survived by his widow, two sons, Edward and Arthur, and one daughter, Alice (Mrs. Harold P. Cowan, of Toronto). There are four grandchildren. Mr. Kennedy failed noticeably in his last year, although he believed that a contemplated trip to California would restore his former vigor. He passed away quite suddenly at his home in Vancouver and was buried in Ocean View Cemetery, leaving to his native Canada a long record of honorable achievement in his chosen career of Pioneer Railway Builder.

Mr. Kennedy was elected a Member of the American Society of Civil Engineers on May 2, 1900.

WILLARD KENT, M. Am. Soc. C. E.*

DIED DECEMBER 22, 1924.

Willard Kent was born on October 27, 1851, at Marshfield, Mass., the son of Warren and Eliza W. (Weston) Kent. His ancestry extends back through various family branches to ten or twelve of the *Mayflower* Pilgrims.

His early life was spent in Marshfield where his father was a Building Contractor and where his primary education was commenced in the public schools. He afterward attended academies at Hanover, Mass., and Woonsocket, R. I.

From 1875 to 1884 Mr. Kent was employed as Chief Engineer in the office of the late John W. Ellis, M. Am. Soc. C. E., Consulting Engineer, of Woonsocket, during which time he managed the engineering work for the Providence and Worcester Railroad Company (now a part of the New York, New Haven, and Hartford Railroad Company), including the design and erection of repair shops at Valley Falls, R. I., as well as several passenger stations.

He was in charge of the design and building of the Woonsocket Water-Works, during 1883 and 1884, on the completion of which he was made Superintendent. In 1884, he opened offices in Woonsocket and Narragansett Pier, R. I., for private practice as a Consulting Engineer and Architect. He remained in charge of the Woonsocket Water-Works, however, until 1890.

Mr. Kent served as Consulting Engineer to the Woonsocket Electric Machine and Power Company, one of the very early municipal electric power plants of New England, the Nourse Mill of Woonsocket being the first textile mill in the United States to be electrically illuminated. He was in charge of the design, construction, and initial operation of the electric light plant at Woonsocket and also of the first electric street car lines in New England—a double trolley system put into operation about 1887.

Mr. Kent was married in 1876 to Jennie Moorhead, of Marshfield, who died in 1893, leaving two sons and a daughter. Later, he was married to Charlotte Angel, of Greystone and Narragansett Pier, R. I., who survives him. He maintained his legal residence in Woonsocket, although he lived principally at Narragansett Pier from 1893 until his death from cancer on December 22, 1924.

He was a member of various Masonic bodies in Rhode Island, as well as of the Boston Society of Civil Engineers and of the New England Water Works Association, having served as Secretary of the latter for many years.

Mr. Kent was elected a Member of the American Society of Civil Engineers on October 2, 1907.

* Memoir prepared by J. Parker Snow, M. Am. Soc. C. E.

PAUL KIRCHER, M. Am. Soc. C. E.*

DIED JULY 10, 1927.

Paul Kircher, the son of the Rev. Johannes G. and Louise C. (Nieman) Kircher, was born in Chicago, Ill., on July 27, 1890.

Mr. Kircher's early education was received under the direction of a tutor, following which he entered the Francis W. Parker School, from which he was graduated in 1908. He then entered the Engineering School of the University of Illinois, but by reason of extra work and diligent study in the Liberal Arts College, he received, in three years, the degree of Bachelor of Arts (1911) and his Bachelor of Science degree in 1912. The degree of Civil Engineer was conferred in 1918 on the presentation of his thesis, the subject of which was "Concrete Ships". While in college, he was a member of Triangle and of Scabbard and Blade (honorary) Fraternities. His height, 6 ft. 2 in., made him an imposing Colonel of the University Regiment. He was also an honor student, having been elected to Sigma Xi Fraternity. In 1910, during his college career, he was employed as Chainman for a City and County Surveyor of Chicago, and the following year as Rodman for the City of Chicago Heights, Ill.

Following graduation from college, Mr. Kircher spent a year traveling in Europe. He then entered the employ of the Illinois Central Railroad Company as a Draftsman, remaining until November, 1916, when he resigned to take employment with the Universal Portland Cement Company of Chicago as an Engineer in the Promotion Bureau. In October, 1917, he became identified with the C. F. Massey Company, and, later, its successor, the Massey Concrete Products Corporation, with which organization he remained until his death. During this period, he served, first, in the capacity of Office Engineer, and, later, as Sales Engineer, Resident Manager, and, finally, Vice-President of the Canadian Concrete Products Company, Limited, a subsidiary, and also as Resident Manager of the Union Switch and Signal Company. For a period of five years, from 1920, he had responsible charge of the operation of the centrifugal concrete pole business of the Company and developed the theory and checked by experiments the designs of hollow reinforced concrete poles and beams.

From 1919 until his death, he also served as a member of the Joint Concrete Culvert Pipe Committee, representing the American Concrete Pipe Association.

Mr. Kircher was a man of high technical capacity, possessing a keen, mathematical mind. He was of great value to his Company and to the committees that he served as a Technical Adviser. His personality also endeared him to all who knew him or came in contact with him.

He was married in October, 1914, to Catherine Planck who, with three children, Paul, Joanne, and Robert Emerson, survives him.

* Memoir prepared by Charles Gilman, M. Am. Soc. C. E.

He was a Licensed Professional Structural Engineer in Illinois and Canada. He was also a member of the Chicago Engineers Club, the Toronto Engineers Club, the Montreal Engineers Club, the Midland Club of Chicago, and the Marlborough Golf Club.

Mr. Kircher was elected an Associate Member of the American Society of Civil Engineers on October 8, 1918, and a Member on June 16, 1924.

BATTLE HARGROVE KLYCE, M. Am. Soc. C. E.*

DIED MAY 23, 1928.

Battle Hargrove Klyce was born in Carpenteria, Calif., on December 10, 1882, the son of a minister of the Methodist Episcopal Church. In his early childhood his parents moved to Tennessee. His primary education was received in the public schools of the State until he became old enough to prepare for college. He then attended the Webb School for Boys at Bellbuckle, Tenn., where he studied under the famous Sawney Webb—founder of the school—who enjoyed the distinction of being one of the outstanding teachers and developers of boys' character in all the Southland. On the completion of his course at the Webb School, Mr. Klyce entered Vanderbilt University, at Nashville, Tenn., enrolling in the course in Civil Engineering; he was graduated from Vanderbilt in 1906 with the degrees of Bachelor of Arts and Bachelor of Engineering. His work in college was of the highest order, winning for him the honor of membership in Phi Beta Kappa. He also won the Founders' Medal for scholarship, the highest honor to be achieved at Vanderbilt University.

Immediately after his graduation, Mr. Klyce was employed as Assistant City Engineer of Jacksor, Miss., which position he occupied for about three years, when he was made City Engineer. He held this position until January, 1911, when he became associated with Solomon and Norcross, of Atlanta, Ga., for which firm he supervised the construction of municipal improvements in several Southern cities. In 1912, he became City Engineer of Waycross, Ga., where he remained for about a year and one-half, after which he returned to private work with the late James N. Hazlehurst, M. Am. Soc. C. E., at Miami, Fla. On the completion of this engagement he was employed by the City of Miami to take charge of the design and construction of various municipal improvements, including sewers, sewage pumping station, sewage disposal works, incinerator, paving, etc. This work extended over a period of three and one-half years.

In June, 1916, Mr. Klyce became associated with W. J. Kackley, M. Am. Soc. C. E., under the firm name of Klyce and Kackley, of Nashville, Tenn., engaged in general engineering business in Florida, Georgia, Tennessee, and Arkansas. Mr. Klyce also acted as Project Engineer for the U. S. Housing Corporation for the proposed housing development at Wrigley, Tenn., and as Supervising Engineer to the Construction Quartermaster for the Aviation Gunnery School at Chapman Field, Miami.

His outstanding work was the design and construction of the Miami Causeway, a structure which extends a distance of three and one-half miles across Biscayne Bay, and which is a monument to his engineering ability.

The partnership of Klyce and Kackley was dissolved in 1920, after which Mr. Klyce continued in business in his own name for several years. In 1923, he became associated with C. N. Harrub, M. Am. Soc. C. E., later changing the firm name to Klyce and Harrub. The new firm continued in municipal

* Memoir prepared by C. N. Harrub, M. Am. Soc. C. E.

engineering work, and at the time of Mr. Klyce's death, was serving many cities in Tennessee, Kentucky, and Mississippi.

In 1912 he was united in marriage to Marion Watkins, of Jackson, Miss., who, with their five children, Erskine, Battle H. Jr., Shelby, Billie, and Mary Alice, survives him.

Soon after Mr. Klyce's return to Nashville from Florida, he connected himself with the Blakemore Methodist Episcopal Church of that city, and became one of its leaders. He was Chairman of the Building Committee for the construction of the new church in 1924, and, subsequently, was also Chairman of the Finance Committee.

Mr. Klyce's varied experience in many States of the South made for him an unusually large circle of friends, as all who knew him liked him and respected him for his manliness and high Christian character. He was a member of the Engineering Association of Nashville, of which he was President in 1927. He was also a Past-President of the Nashville Local Section of the Society. He was a member of the Nashville Automobile Club and the Lions' Club.

Mr. Klyce enjoyed the respect and esteem of his engineering colleagues, and his death is a blow to the profession which will long be felt; his associates in church and business will find his place difficult to fill.

Mr. Klyce was elected an Associate Member of the American Society of Civil Engineers on June 30, 1911, and a Member on March 9, 1920.

OSCAR FRANCIS LACKEY, M. Am. Soc. C. E.*

DIED DECEMBER 19, 1928.

Oscar Francis Lackey was born in Washington, D. C., on July 1, 1874. He was the son of the late Milford Fiske Lackey, who served for many years as Chief Auditor of the Treasury Department. The family moved to Baltimore, Md., when Oscar was thirteen years old. He was educated at Rock Hill College and was graduated from Johns Hopkins University in 1896. In 1897, he went to Santiago, Chile, as an Engineer of the War Department under the late Gen. Leonard Wood, U. S. A. He survived an attack of yellow fever and returned to Cuba on dock construction work and, later, went to Panama on an engineering engagement connected with the construction of the Panama Canal.

Mr. Lackey returned to the United States in 1905 and was appointed Principal Assistant to the late N. H. Hutton, M. Am. Soc. C. E., who was then Chief Engineer and President of the Harbor Board of Baltimore. When Major Hutton died, a few years later, Mr. Lackey succeeded him in this important post and had charge of important dock and pier construction. He was one of the first, if not the first, engineer to utilize reinforced concrete piles in pier construction. He held this position until 1915, and his extensive harbor development plans and recommendations for a $50 000 000 loan for this purpose were looked upon as fanciful at the time. Later developments, since the end of the World War, have thoroughly justified Mr. Lackey's vision, and a loan of this amount has been passed by the Maryland Legislature and voted by the people.

In 1912, Mr. Lackey was selected as one of the five Directors of the Association of Seaport Authorities and, in 1914, he was elected President of the Association of Port Authorities.

In 1915, he became Assistant to the President of the Poole Engineering Company, of Baltimore, and in this position negotiated the purchase of a manufacturing plant in Lafayette, Ind., engaged in the manufacture of shells for the British Government; he operated the plant for two years. In 1918, he was appointed Supervising Engineer in the War Department, under the late Maj.-Gen. George Goethals, U. S. A., M. Am. Soc. C. E., for the construction of the Atlantic Port terminals in Boston, Mass., Philadelphia, Pa., Brooklyn, N. Y., Norfolk, Va., New Orleans, La., and Charleston, S. C.

Mr. Lackey returned to Cuba on consulting engineering work in 1921 and, in 1922, he went to Colorado Springs, Colo., as Vice-President and General Manager of the Colorado Springs Light, Heat, and Power Company. On the change of ownership of this Company, he returned to the East and was engaged as Consulting Engineer on several dock and pier projects. In 1924, in addition to his consulting work, he was appointed head of the Transportation Bureau of the City of Baltimore, which position he held

* Memoir prepared by Ezra B. Whitman, M. Am. Soc. C. E.

until 1927. Shortly before his death, he was engaged by the State Roads Commission of Maryland to pass upon the plans of the proposed $10 000 000 bridge across Chesapeake Bay.

While in Panama, Mr. Lackey was injured on the work, and although this injury troubled him until the time of his death, his indomitable spirit kept him going when men of lesser courage would have given up. He was of the stuff that never yields to misfortune, but turns defeat into victory. He was intensely loyal to his friends and was ready at all times to fight for what he believed to be right. His uncompromising spirit, his undeviating loyalty to high personal principles and to professional standards marked him as a man in whom others could put their trust. To his professional work, he gave not only his head, but his heart, and whatever he did was done with all his might. He was never lukewarm nor a slacker in anything, and he never failed to make good where it was humanly possible to do so.

Mr. Lackey died in Baltimore on December 19, 1928, of pneumonia, after an illness of less than a week. His death was a great shock to his many friends and came only a few weeks after his appointment by Governor Ritchie to the important position as Chairman of the State Roads Commission of Maryland.

Mr. Lackey was elected a Member of the American Society of Civil Engineers on April 6, 1909.

CHARLES WELLFORD LEAVITT, M. Am. Soc. C. E.*

DIED APRIL 22, 1928.

Charles Wellford Leavitt, the son of C. W. Leavitt and Sarah Allibone Leavitt, was born at Riverton, N. J., on March 13, 1871. He was educated at the Gunnery School, at Washington, Conn., and at the Cheltenham Military Academy of Pennsylvania.

Mr. Leavitt's engineering career began in 1888, when he accepted a position as Assistant Superintendent of Construction with his father's contracting firm. A year or so later, he became Assistant Engineer of the Caldwell, N. J., Railroad and was subsequently promoted to the position of Chief Engineer.

In 1892, he became Chief Engineer of the New York Suburban Land Company, engaged in the development of Essex Fells, N. J. Mr. Leavitt remained with this Company for three years, finally leaving it to enter the employ of the East Jersey Water Company.

In the fall of 1896, Mr. Leavitt formed a partnership with Rudolph Ulrich, a Landscape Architect, well known for his abilities. This partnership continued for about two years, at the end of which time Mr. Leavitt opened an office and established himself as a Civil and Landscape Engineer, concentrating on this work, which gave him so much pleasure and held so much of interest during his lifetime. A review of some of the outstanding results of Mr. Leavitt's life work shows how diversified were his attainments within the scope of his field.

Among the larger private estates which were developed under his guidance were those of William C. Whitney, Daniel Lamont, Foxhall Keene, and Charles M. Schwab.

Belmont Park, Saratoga, Empire City, Havre de Grace, and the Montreal Jockey Club are race-tracks on which Mr. Leavitt's planning had a controlling influence.

He made plans for the Palisades Interstate Park; Stamford Park at Stamford, Conn.; Monument Valley Park, at Colorado Springs, Colo.; Schenectady Park, Schenectady, N. Y.; Mahlon Stacy Park and the State House Park, at Trenton, N. J.; Saratoga Springs Park, Saratoga, N. Y., and for the Fairmount Park Commission of Philadelphia, Pa. He was also Landscape Engineer for the Camden County, New Jersey, Park Commission.

Among the Country Clubs which were developed under Mr. Leavitt's landscape plans were the Westchester-Biltmore, Rye, N. Y.; the Rumson Country Club, Rumson, N. J.; the Englewood Golf Club, Englewood, N. J.; the Lido Country Club, Long Beach, Long Island, and the New Orleans Country Club, New Orleans, La.

Many large real estate developments were planned and supervised by Mr. Leavitt, among which may be mentioned those for the Pennsylvania Steel Company at Steelton, Pa.; for the Worth Brothers, at Coatesville, Pa.; the Kilgour and Stettinius Estates, at Cincinnati, Ohio; Milner Heights, Birmingham, Ala.; and Long Beach, Long Island, N. Y.

* Memoir prepared by M. W. Weir, M. Am. Soc. C. E.

University and institutional grounds also attracted his interest, and among those which he laid out were: Tome Institute; the New York Juvenile Asylum; the Peabody College for Teachers; Oglethorpe University; the Presbyterian College of South Carolina; Lehigh University; and Fordham University.

Among the cemeteries which Mr. Leavitt planned are the Gate of Heaven Cemetery, at Kensico, N. Y., and part of Woodlawn Cemetery, at Woodlawn, N. Y.

His city planning practice also included work done for Camden, N. J.; Garden City, N. Y.; Trenton, N. J.; Brunswick, Ga.; West Palm Beach, Fla.; Lakeland, Fla.; and Mount Vernon, N. Y.

Mr. Leavitt was a member of the American Society of Landscape Architects, Past-President of the American Institute of Consulting Engineers, a member of the Engineering Committee for the Plan of New York and Its Environs, the Architectural League, the National Conference on City Planning, the American City Planning Institute, the Interstate Metropolitan Planning Conference, the International Garden Cities and Town Planning Federation, the National Conference on State Parks, and the Garden Club of America.

He was a member of the Union Club, the Transportation Club, the Westchester-Biltmore Country Club, the Racquet Club, Philadelphia, the Adirondack League Club, the Play and Players Club, Philadelphia, and the New England Society of New York.

He was also a member of the New York State Chamber of Commerce, the Westchester Chamber of Commerce, the American Civic Association, and the New York State Board of Real Estate Boards.

Mr. Leavitt was a man of great energy and his interest in public affairs was keen, although he took no part in politics. Of clear mental vision, his judgment was sage and well founded. His imagination could always be depended upon to look forward, and from this quality, coupled with business acumen and a driving personality, came, in a large measure, the successes for his clients in his chosen profession.

With these attributes, his congeniality, and his ready wit, Mr. Leavitt was a delightful companion, a splendid friend, and an inspiring leader. The influence of such a life cannot be estimated, but of this one may be sure, it was great, and its reflection will persist for the good of all.

He was married to Clara Gordon White on September 26, 1899, and is survived by his widow and four children, Gordon, Kent, Charlotte, and Dundas.

Mr. Leavitt was elected an Associate Member of the American Society of Civil Engineers on January 4, 1899, and a Member on May 2, 1905.

FRANK McDOWELL LEAVITT, M. Am. Soc. C. E.*

DIED AUGUST 6, 1928.

Frank McDowell Leavitt was born at Athens, Ohio, on March 3, 1856. He was the second son of the Rev. McDowell Leavitt and Bithia (Brooks) Leavitt. His paternal grandfather, Humphrey Howe Leavitt, as a young lawyer, had moved from Eastern Massachusetts to Southern Ohio and later became, in turn, Congressman for the Cincinnati District and Judge of the United States District Court. His maternal grandfather, Moses Brooks, when a young man, had moved from Western New York to Cincinnati, Ohio, where he became a substantial merchant and then a prominent member of the bar of that city.

In 1868, Mr. Leavitt's father moved to New York, N. Y., and subsequently became President of St. John's College, at Annapolis, Md., and of Lehigh University. On arrival in New York, young Frank was entered in the famous Hunter Public School on 13th Street, but, in 1869, the lad was forced to leave because the family moved to Orange, N. J. After two years of school in Orange during which he had shown the most pronounced predilection for engineering, and decided qualities of self-discipline, industry, and self-reliance, he entered Stevens Institute of Technology from which he was graduated in 1875. His Alma Mater conferred on him the degree of Mechanical Engineer in 1905, and the degree of Doctor of Engineering, on the Fiftieth Anniversary of the Institute, in June, 1921.

Immediately after graduation, Mr. Leavitt entered the employ of the late Fred E. Sickels, M. Am. Soc. C. E., a prominent engineer of that day. One of his first assignments was the development of a steam steering gear for the U. S. S. *Trenton*, which vessel was lost in the great Samoan hurricane about 1888.

In August, 1876, he became Head Draftsman for Bliss and Williams, of Brooklyn, N. Y., and was employed there until early in 1881. He then decided to try other branches of engineering and for nearly a year was Master Mechanic of the Texas Division of the Mexican National Railroad, with headquarters at Corpus Christi, Tex. Late in 1881, he returned to New York and became Superintendent of the Graydon and Denton Manufacturing Company, of Jersey City, N. J., manufacturers of rock drills and mining machinery. He remained in this work for two years and patented a number of important improvements in the tools he was then manufacturing.

In 1883, however, Mr. Leavitt withdrew from the mining machinery field and returned to the E. W. Bliss Company (successor to Bliss and Williams), as Assistant Superintendent, and, in 1884, was appointed Superintendent of that Company. From this time until about 1900, he was actively engaged in designing and manufacturing a most varied line of sheet-metal working machinery.

* Memoir prepared by Logan Cresap, Commander, U. S. N. (*Retired*), New York, N. Y.

The records of the Patent Office commenced to show his name as an inventor within a year or two of his graduation from Stevens Institute, and from that time until 1921 or 1922, a constant stream of patents aggregating approximately three hundred, testify to his mechanical skill and inventive genius. The outstanding accomplishment of the early years of this productive life was the invention and development of the first automatic tin can body-making machine. Until that time, such tin cans as were used in the United States were made practically by hand and to Mr. Leavitt belongs the honor and distinction of having been the first man to conceive and develop a fully automatic machine which would perform all the necessary operations in the production of such can bodies. At that time the rate of speed at which this machine made these bodies was considered tremendous and so far beyond the normal requirements of can users that much difficulty was encountered in selling it. Meeting little or no encouragement in this country the machine was sent to the Paris Exposition about 1880 in the hope that some of the large canners of England or Continental Europe would appreciate its economic value. Unfortunately, however, only failure met this effort and the machine had to be brought back to Brooklyn and for a time was regarded practically as a "white elephant". Finally, however, one of the large can makers and packers in the Chicago District became interested and purchased it. Its use gave a tremendous impetus to canning in America and very properly may be regarded as the first stage in the modernization of that industry.

Another machine of Mr. Leavitt's which soon came into wide and ever-expanding use was the toggle drawing press for use in the manufacture of all sorts of hollow drawn ware, kitchen utensils, etc. His early inventions in drawing and stamping machines were the initial steps in the development of the huge power presses used to-day, particularly in this country, in the automobile industry for the quantity production of automobile bodies, frames, and accessories.

In addition to these two specific machines, on which improvements naturally were made later, Mr. Leavitt patented many original types of other machines, such as, drop hammers, punching presses, clutches, air compressors, shaft couplings, pumps, etc.

In the late Eighties, the Navy Department was making efforts to introduce into the United States the manufacture of up-to-date ordinance. These efforts resulted in the purchase of many European patent rights by American firms and the introduction of the manufacture of the latest types of armor-plates, armor-piercing projectiles, built-up guns, and automobile torpedoes into American industry. The Bethlehem Steel Company, the Carpenter Steel Company, and the E. W. Bliss Company were among the important commercial pioneers in this movement.

After an extensive tour abroad, during which he visited the British, Austrian, and German torpedo and projectile manufacturing plants, Mr. Leavitt, representing the E. W. Bliss Company, purchased the American rights to the Whitehead torpedo and, in 1890, introduced its manufacture into America. To the surprise of the vendors, he purchased no patent rights

or machines involved in the complicated manufacture of the torpedo, but immediately on his return, set to work to develop his own manufacturing equipment. In this, he was so successful that the original manufacturers were soon outstripped in economical and accurate production. During this same period he designed and installed the plant of the United States Projectile Company, in which small and medium caliber "common" projectiles were manufactured.

Subsequent to 1890 Mr. Leavitt's activities were gradually shifted from metal-working machines to automobile torpedoes and from 1900 until just prior to the World War, the Patent Office records show a steady stream of developments in torpedoes under his name. When he first undertook the manufacture of the Whitehead torpedo, it was a puny bronze instrument about 11 ft. long and 17 in. in diameter, with a range of about 800 yd., uncertain directive control, and carried an explosive charge of about 50 lb. of gun cotton. Mr. Leavitt gradually improved the torpedo by the introduction of steel for the air flask, the Curtis turbine for propulsion, the combustion of fuel in the air supply, the generation of steam in connection with that combustion, an entirely new and dependable gyroscopic steering apparatus, and many other innovations. His development of the gyroscope, alone, was an important contribution to mechanical engineering, and found application in other fields.

In order to secure the time to assemble and properly develop his ideas for this new torpedo, he left the E. W. Bliss Company shortly after 1900, and for a year or more established his own office and drafting-room in New York. On the issue of the patents for this new torpedo and their purchase by the E. W. Bliss Company he returned to that Company as an Officer and Director. This new torpedo became known as the Bliss-Leavitt torpedo and from about 1910 until the present time it has been, and still is, the standard equipment of the United States Navy. At the beginning of the World War this torpedo was about 21 ft. long and 21 in. in diameter, capable of reliable and accurate control over a range of about 13 500 yd. It was propelled by a super-heat steam and combustion gas turbine, operating at a temperature of about 1 000 to 1 100° Fahr., the most efficient torpedo in any navy, and carrying an explosive charge of more than 200 lb. of tri-nitro-toluol.

At the outbreak of the World War Mr. Leavitt offered his services to the Navy Department and shortly thereafter was placed in charge of a small committee known as the "Committee on Experimental Power", to which the Bureau of Steam Engineering assigned the task of developing a steam plant for the propulsion of aircraft. For this work, Mr. Leavitt was commissioned on a dollar-a-year basis. Having by this time practically retired from active connection with the E. W. Bliss Company, except as Director, he could devote his entire time to this work, which he proceeded to do with his usual vigor. At the end of the war the Navy Department decided to continue the work of this Committee, as it was believed that although the boiler plant which had been developed might not actually become a competitor of the gasoline engine, for aircraft use, such promising and interesting features had been produced or, were under investigation, which possibly might have a much

wider application in naval engineering, particularly in submarine propulsion, as to warrant the continuation of the experimentation. The Committee, therefore, was reduced in size and was authorized to complete its investigation under Mr. Leavitt's direction. It continued to function for nearly two years, and when finally dissolved, turned over to the Navy Department plans and test records for an aircraft boiler which, with all appurtenances, control apparatus, and the necessary water, delivered about 1 000 h. p., with a total weight of about 2 000 lb. This was practically the last engineering work which Mr. Leavitt undertook.

He was known in the Engineering Profession, and particularly among his friends and associates in that profession, as a genius in conceiving and designing mechanism to accomplish, with the utmost simplicity, a given purpose or necessity. His remarkable ability to visualize a problem, and his knowledge of mechanical motion and force, enabled him to picture the most difficult mechanism in the simplest of forms, to design its parts in detail, and to direct its construction in the shop, ofttimes without even assembling the parts on drawings.

Mr. Leavitt was a gentle, modest, and friendly man, simple in his tastes and personal requirements, and these qualities endeared him to those who knew him personally, and claimed, and held, the respect of his business associates. His principal diversions were painting in oils and reading. He was an omnivorous and broad reader, possessing the happy faculty of retaining what he read, and the profundity of his knowledge, when unconsciously revealed, was a source of wonder and enjoyment to his friends. In his personal relationships, his outstanding quality was a kindness and willingness to assist others, with that help which relieves the necessity, and the manner that graces the act.

In 1893, Mr. Leavitt was married to Gertrude Mitchell Goodsell, of New York, the daughter of Charles Morgan and Elizabeth Mitchell Goodsell. He is survived by his widow and a daughter, Beatrice Leavitt.

He was a member of the American Society of Naval Architects and Marine Engineers, the American Society of Mechanical Engineers, and the Engineers' Club of New York, the University Club of Washington, D. C., the Riding and Driving Club, and the Montauk Club, of Brooklyn, the Scarsdale Golf Club, of Scarsdale, N. Y., and the Sankaty Head Golf Club, of Siasconset, Mass.

Mr. Leavitt was elected a Member of the American Society of Civil Engineers on December 2, 1885.

GUSTAV LEHLBACH, M. Am. Soc. C. E.*

DIED MARCH 26, 1926.

Gustav Lehlbach was born in Baden, Germany, on July 3, 1844. The son of a Lutheran minister of progressive tendencies—a man who thought of principle first and safety afterward—he was brought to this country by his parents when, with Carl Schurz and Baron Friedrich von Steuben, they were forced to flee from Germany on account of his father's active participation in the rebellion of 1848. The family settled in Newark, N. J., where the Rev. Mr. Lehlbach accepted the pastorate of the old Mulberry Street German Evangelical Church.

Gustav Lehlbach attended the public schools in Newark and then started his preparation for an engineering career in the old way—by apprenticeship to a successful engineer. This connection was severed temporarily when, at the age of sixteen, at the outbreak of the Civil War, he marched off as a volunteer in General Kearney's unit. The pioneering spirit of his forebears found early expression when, one cold March morning, Mr. Lehlbach, with a single comrade, swam the Rappahannock River stripped to the skin, in order to be the first in the enemy's rifle-pits. Fortunately, the two adventurers were right in their conjecture that the Confederates had abandoned the pits during the night.

In 1861 he was severely wounded in his first engagement at the Battle of South Mountain. On his recovery he was assigned to the Signal Corps, and, toward the close of the war, was stationed as one of President Lincoln's bodyguard at Arlington Headquarters. Whether from the contacts of that period, or from subsequently acquired appreciations, he, like so many others on both sides of the Mason and Dixon Line, came to have an admiration for Abraham Lincoln which grew to reverence deep enough perhaps to have had something to do with later attitudes and ideals. Particularly in his later judgments of men in public life, and in the formation of his own views on political, social, and religious subjects, Mr. Lehlbach was much influenced by Lincoln—and by Jesus of Nazareth. He saw the Nazarene as Lincoln saw Him—clearly, undistorted by the refracting media of church, sect, and dogma.

After General Lee's surrender at Appomattox, Mr. Lehlbach took up his engineering apprenticeship where he had left it in 1861. In two short years he was Resident Engineer, locating and constructing the Newark and New York Railroad, not then a branch of the New Jersey Central, but an independent project backed by Newark men. Owing to his foresight, no grade crossings over Newark's streets were permitted, except at Mulberry Street, and this was eliminated later. He also sought to have the line carried at a high level over Broad Street to tap the Oranges, but in this he was overruled.

Bridge design in those days was a very much simpler matter than it is now; so that by continuous hard work, made possible by an exceptionally sturdy physique, Mr. Lehlbach acquired a working knowledge of that subject,

* Memoir prepared by Henry G. Babcock, Assoc. M. Am. Soc. C. E.

and, if the writer's recollection serves him rightly, was thus enabled to design the bridge across the Passaic River. The caisson foundations for the center pier of this old swing-bridge were, it is believed, his choice of construction method and were sunk under his direction.

From 1869 until the fall of 1871 he was Chief Engineer on the location and construction of the Port Royal Railroad from Augusta, Ga., to Port Royal, S. C., and subsequently Resident Engineer on the construction of the New York and Long Branch Railroad. On April 19, 1872, he became, as City Surveyor of Newark, the head of the Engineering Department of that city.

For five years, from 1875 to 1880, Mr. Lehlbach was engaged in private practice as a partner in the firm of Lehlbach Brothers at 196 Market Street, Newark, a business still carried on at the same address by his nephews, Herman B. and Milton Lehlbach.

In 1880, came another chance for that railroad pioneering which enlisted his enthusiasm more than any other engineering activity. First, in this period, came the Dover and Rockaway Railroad for the Central Railroad Company of New Jersey. Then, about 1883, he located the line of the Denver and Rio Grande Western Railroad, in Utah. Accurate information covering this part of Mr. Lehlbach's life is meager, but probably there should be assigned to it an episode in itself well enough authenticated. While engaged in reconnaissance, Mr. Lehlbach covered many miles on horseback and unaccompanied. Many of these lonely expeditions were through uninhabited desert, and on one of them he unwittingly drank some of the poisonous alkaline water in which those deserts abound. Soon taken violently ill and unable to ride farther, he dismounted and lay helpless on the sand through two or three days, each of which must have seemed weeks, of the burning desert heat, while his horse foraged at will, presumably with very little success. The attack finally wore off. He recovered his horse and rode back into camp.

Similar episodes attended his work in Mexico where he acted for the Mexican Government, once more on railroad reconnaissance. Although his party numbered 30 or 40 men, Mr. Lehlbach, as usual, made many solitary expeditions. On one of these occasions, in attempting to ford a stream which he had traced for 30 miles, his horse suddenly struck quicksand. By grasping the overhanging limbs of a tree, he was able to pull himself to safety and then to coax and encourage his horse to struggle back to *terra firma*. Unable, on account of the quicksand, to retrace his steps, he seemed to be facing another desperate situation, as he knew of no settlement within a hundred miles. He gave his horse the bridle and the creature's instinct brought him to a trapper's cabin where he found food and shelter for the night, and whence he eventually found his way back to camp.

While in Mexico Mr. Lehlbach experienced real Mexican hospitality in the home of General Bustamante. During the Civil War the General had been the delegate in Washington of the Mexican patriots who were trying to oust Maximilian, and were seeking President Lincoln's help. President Lincoln, as is well known, could not see his way clear to give material assistance or even recognition and open moral support to the enemies of Maximilian. Yet, he

evidently left no doubt in Bustamante's mind as to where his personal sympathies lay—and he must have left an indelible impression of his personality on the General. Perhaps, when Mr. Lehlbach asked General Bustamante to describe that impression, he was, in memory, mounting guard again at Arlington. Certain it is, however, that the Mexican General's reply found many an audience in Mr. Lehlbach's later days when railroad reconnaissance had become reminiscence. Said Bustamante, "He was another Jesus Christ."

Mr. Lehlbach was married to Anna Lydia Babcock, of Communipaw, N. J., on January 17, 1872. During his married life his engineering business came to take the place of railroad pioneering, and he eventually settled down in Newark where he ended his days. Toward the close, some visitors to his Newark home were perhaps curious about a sort of double-decked picture frame which hung from the library mantle. The lower story enclosed a portrait of President Woodrow Wilson for whom Mr. Lehlbach had acquired a warm admiration; but from the upper story of the frame, above the War President of 1917, looked out the face of the War President of 1861. The visitor who remarked about that double-decked portrait was quickly made to feel how much it meant to Mr. Lehlbach. It evidently had to do with the boyhood days at Arlington, with dreams and ideals revivified for him by the words and acts of the original of the lower portrait.

He died on March 26, 1926, his widow surviving him by less than two years. He had no children.

Mr. Lehlbach was elected a Member of the American Society of Civil Engineers on March 7, 1883.

EMILE LOW, M. Am. Soc. C. E.*

DIED MAY 25, 1927.

Emile Low, the eldest son of Sigismund and Anna (Hussell) Low, was born on April 2, 1854, in Pittsburgh, Pa. His father was a graduate of the Karlsruhe Polytechnicum, Karlsruhe, Germany, and was one of the founders of the *Pittsburgh Volksblatt.* Emile Low's education was obtained in a private school and, later, in the public schools of Pittsburgh.

His first professional work was done in 1869 in connection with the surveys of the Monongahela Valley Railroad. In 1870, he was engaged on the surveys for the Western Extension of the Pittsburgh and Connellsville Railroad (now a part of the Baltimore and Ohio Railroad), of which his father was Engineer in Charge; and during 1870 and 1871, he was employed on construction work for the Cumberland, Md., extension of the same line.

Subsequently, Mr. Low became Assistant on the Hiland Avenue Reservoir of the Pittsburgh Water-Works, with which he was connected until 1875. In 1876, he served as Assistant City Engineer. Early in 1877, he was connected with the survey of the Pittsburgh and Lake Erie Railroad Bridge across the Ohio River. Later, he entered the service of the Pittsburgh, Castle Shannon, and Washington Railroad Company as Principal Assistant Engineer; this railroad subsequently was absorbed by the Baltimore and Ohio Railroad Company. In 1879, he was engaged on work for the Shousetown Railroad and also for the Pittsburgh and Atlantic Railroad Companies.

In the fall of 1879, Mr. Low was appointed Engineer of Construction and of Maintenance of Way for the Pittsburgh and Western Railroad Company and, later, became Chief Engineer. This railroad, also, became a part of the Baltimore and Ohio Railroad. He remained with the Company until March, 1881, when his father was appointed Engineer in Charge of the Guanajuato Branch of the Mexican Central Railway, and Mr. Low accompanied him to Mexico as his Assistant. This work terminated in June, 1882. During the pursuit of his engineering career, Mr. Low encountered many dangerous situations, particularly that experienced in a silver mine in Mexico where he barely escaped with his life because of a dynamite blast discharged by careless workers.

Upon his return to the United States, he was appointed Engineer in charge of location for the Pittsburgh, McKeesport and Youghiogheny Railroad Company with which he remained until the latter part of 1884. Early in 1885 he served as Assistant Engineer with the Monongahela Railroad Company of Pittsburgh, which position he retained until March, 1885, when he was made Locating Engineer of the Pennsylvania (Schuylkill Division) Railroad. In December, 1886, he made the remarkable location of a mountain line, 20 miles long, in 10 days during winter weather, between Delano and Hazelton, Pa.

In 1887, Mr. Low was appointed Locating Engineer of the Norfolk and Western Railroad. Subsequently, he became Division Engineer and Engineer

* Memoir prepared by H. J. March, L. L. Davis, and S. T. M. B. Kielland, Members, Am. Soc. C. E.

in charge of coal operations of the Ohio Extension. In 1892, he was made Engineer of the Mathieson Alkali Works, of Saltville, Va. The late John F. Wallace, Past-President, Am. Soc. C. E., who was Vice-President of the Works, took occasion at a speech-making event to commend most highly the accuracy and thoroughness of Mr. Low's work. He stressed the thought that a great many engineers, like Mr. Low, spend their lives in obscure sections of the country and in consequence nothing praiseworthy is heard of them; therefore, when opportunity presents itself for commendation, it should be given freely. Mr. Low's connection with the Mathieson Alkali Works terminated in 1896, after which he became associated with Maj. Thomas W. Symons, Corps of Engineers, U. S. Army, in the preparation of a report on the Erie Canal. He also was associated with Major Symons in the preparation of the well-known report for a canal from the Great Lakes to the Hudson River.*

From 1897 to 1903, Mr. Low was Resident United States Assistant Engineer, in charge of the building of the Buffalo Breakwater under the direction of Colonel Symons. He was awarded the Norman Medal by the Society in 1904 for the presentation of a paper describing this work.†

Mr. Low continued as U. S. Assistant Engineer until June 1, 1904, after which he was in the service of the State of New York as Resident Engineer in charge of the Barge Canal work in Oneida County, having been granted a three-year furlough from the U. S. Engineer Department. While he was engaged in this work which involved some difficult engineering problems, he published a number of interesting papers, one of which was entitled "Wash-drill Borings in the New York State Barge Canal",‡ and another, "Hydraulic Dredging on New York State Barge Canal".§ On April 1, 1907, Mr. Low re-entered the Federal Service as Assistant Engineer at Buffalo, N. Y., where he remained until his retirement in April, 1924, having served under the following officers of the Corps of Engineers, U. S. Army: Col. H. M. Adams, Col. W. L. Fisk, Col. J. G. Warren, M. Am. Soc. C. E., Maj. L. V. Frazier, Col. Clarke S. Smith, M. Am. Soc. C. E., Maj. William E. R. Covill, and Maj. P. S. Reinecke, M. Am. Soc. C. E.

Mr. Low was married on January 23, 1883, to Estelle Reed, of Zelienople, Pa.; there were three children, Estelle Low, Emile Reed Low, and Mary Virginia Low.

Mr. Low was esteemed most highly by his fellow engineers of Buffalo for his remarkable professional attainments. He was an indefatigable worker in every engineering pursuit which he followed. In this respect he was an exemplary character for so many engineers who take joy in service for their fellow men and their country and who contribute of their energy of mind, body, and soul, as it were, with little or no personal reward.

Mr. Low was elected a Member of the American Society of Civil Engineers on May 2, 1888.

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* The respective reports upon these matters may be found in the Annual Report of the Chief of Engineers, U. S. Army, for 1897.

† "The Breakwater at Buffalo, New York," *Transactions*, Am. Soc. C. E., Vol. LII (1904), p. 73.

‡ *Engineering News*, January 17, 1907.

§ *Loc. cit.*, April 10, 1913.

FRANK OSCAR MAXSON, M. Am. Soc. C. E.*

DIED JANUARY 21, 1928.

Frank Oscar Maxson was born in Stillmanville, Conn., on August 8, 1851, of pioneer stock. When he was three years of age his family, influenced by the Western movement of the times, went to California, making the journey by the long circuitous route of the Isthmus of Panama. Eight years were spent in the mining districts of California, and Mr. Maxson's two younger brothers and a sister were born there. In 1862, however, ill health compelled his mother to return to her home in Rhode Island, and the four children accompanied her.

After his mother's death, Mr. Maxson and his brother, Louis, "boarded out", helping to support themselves by carrying papers before school in the mornings, the younger children being cared for by relatives. The two older boys attended the public schools of Norwich, Conn., and, later, the Norwich Free Academy; Mr. Maxson then entered the Sheffield Scientific School of Yale University, at New Haven, Conn. Later, he returned to the Norwich Academy and taught for one year, after which he re-entered Yale for a year of post-graduate work.

After he had completed his studies, Mr. Maxson began his long and faithful service to the United States Government, as Topographer of a party engaged in surveying the States of Colorado, New Mexico, and Arizona. The work also included a survey of the Yosemite Valley in California, and Mr. Maxson made one of the earliest authentic maps of that beautiful and far-famed region. He continued in this work for seven years, half of each year being spent "in the field", the other half in Washington, D. C., plotting and computing the data which he had acquired.

In October, 1881, Mr. Maxson successfully passed an examination for appointment to the Corps of Civil Engineers, U. S. Navy. He was commissioned on October 29 of that year and assigned to duty at the Navy Yard, Pensacola, Fla. He served successively at the New York, Boston, Mass., Washington, and Mare Island, Calif., Navy Yards. During the early days of the war with Spain he was sent to the Norfolk, Va., Navy Yard where he served during the war period. He attained the rank of Lieutenant Commander in 1898, Commander in 1901, and Captain in 1906. Following the close of the war with Spain, Captain Maxson returned to the Boston Yard.

In 1901, he was ordered to the Philippine Islands to take charge of public works and public utilities at the Naval Stations at Cavite and Olongapo. He returned to the United States early in 1904 and was detailed to the League Island Navy Yard at Philadelphia, Pa. Following subsequent duties at Portsmouth, N. H., and Key West, Fla., he was transferred to the retired list with the rank of Captain in August, 1913. In 1915, he was recalled to active duty and detailed to assist the Department of Justice in Washington in the prosecution of several lawsuits involving Navy contracts. He completed this duty in February, 1917, and reverted to an inactive status on the retired list.

* Memoir prepared by DeWitt C. Webb, M. Am. Soc. C. E.

The outbreak of the World War made it necessary again to recall Captain Maxson to active service. In October, 1917, he was placed in charge of all Naval public works in the Seventh Naval District, with headquarters at Key West. He performed meritorious service in this capacity, having charge of the construction of many important Naval war-time projects, including quay walls, piers, barracks, hospital buildings, and the Air Station, at Miami, Fla. He was relieved from active duty in October, 1919, and shortly thereafter moved to Coupeville, Wash., where he resided at the time of his death. He contracted pneumonia about the middle of January, 1928, and passed away after a few days' illness.

Captain Maxson's career as a civil engineer was praiseworthy. All his work exemplified his careful foresight and professional attainments. His patriotism and generous consideration for his fellow men were outstanding characteristics. Any undertaking for the welfare of his country or community received his whole-hearted support.

On December 26, 1877, he was married at Washington, D. C., to Evelyn M. Van Doren. He is survived by his widow, two sons, Dr. Frank T. Maxson and Dr. Louis H. Maxson, of Seattle, Wash., and one daughter, Evelyn Maxson, of Coupeville, Wash.

His home life was ideal and continued for over half a century. On December 26, 1927, the golden wedding anniversary of Captain and Mrs. Maxson was celebrated at their home in Coupeville, at which all the members of the immediate family were present.

Captain Maxson's natural gifts combined with the wide range of his experience and reading made him an interesting conversationalist on a great variety of subjects. He had a most agreeable personality and a keen understanding of human nature. These traits, together with a delightful sense of humor which was habitually expressed in good-natured banter, deeply endeared him to all.

For many years Captain Maxson was an Elder of the Metropolitan Presbyterian Church, of Washington, D. C., and at the time of his death was a Trustee in the Coupeville Congregational Church. He also taught large Sunday School classes in Washington, Key West, and elsewhere. He was not a man whose religious spirit was confined to the church, but rather one who carried it with him into his everyday life, as shown by his innumerable kind deeds and generous service for others.

Captain Maxson was elected a Member of the American Society of Civil Engineers on May 1, 1889.

JAMES C. NAGLE, M. Am. Soc. C. E.*

DIED APRIL 6, 1928.

James C. Nagle was born in Richmond, Va., on October 9, 1865, the son of John and Ellen Mary (Smith) Nagle. His parents moved to Travis County, Texas, where his childhood was spent on a farm, and his primary education was obtained in a country public school near Austin. He received the degree of Bachelor of Science in Civil Engineering from the University of Texas in 1889, and the degree of Master of Arts in 1892. The same year he obtained the degree of Civil Engineer from the University of Pittsburgh and the degree of Master of Civil Engineering in 1893 from Cornell University.

Mr. Nagle's first professional experience was as Field and Office Assistant for the St. Louis Southwestern Railway Company in 1887. He was Assistant Engineer for the Austin and Northwestern Railway Company in 1888. In 1889 and 1890 he was employed as Assistant Geologist, chiefly on topography for a geological survey of Texas.

In 1890, he became Professor of Civil Engineering at the Texas Agricultural and Mechanical College and, in 1911, was appointed Dean of the School of Engineering.

In 1913 Mr. Nagle resigned this position to accept an appointment on the Texas State Board of Water Engineers. He served as the first Chairman of this Board from September, 1913, to September, 1917, when he resigned to return to the Texas Agricultural and Mechanical College as Professor of Civil Engineering, Dean of the Engineering School, Director of the Texas Engineering Experiment Station, and Consulting Engineer for the Prairie View State Normal and Industrial College.

Mr. Nagle served the Texas Agricultural and Mechanical College in these positions until September, 1922, when he resigned to devote his entire time to the business of the Nagle, Witt, Rollins Engineering Company, which he had organized in 1919 and of which he became the Senior member. He was still an active member of this firm at the time of his death.

From 1923 to November, 1927, Mr. Nagle was associated with R. A. Thompson, M. Am. Soc. C. E., on the investigation, design, and construction of a $5 000 000 water supply reservoir for the City of Dallas, Tex.

In 1903, Mr. Nagle was married to Emily St. Pierre Davis (*née* Morgan), at Union, S. C. He is survived by his widow, two step-daughters, Mrs. Robert J. Potts, of Waco, Tex., and Mrs. F. F. La Roche, of Atlanta, Ga., a step-son, Major John F. Davis, U. S. A., of Fort Leavenworth, Kans., and two brothers, Judge M. Nagle, of El Paso, and John Nagle, of Manor, Tex.

In addition to numerous professional and technical papers, Mr. Nagle was the author of a "Field Manual for Railroad Engineers"; of three published reports on the "Silt-Carrying Capacity of Texas Streams"; and of *Bulletin 222*, on "Irrigation in Texas", of the United States Department of Agriculture.

* Memoir prepared by A. P. Rollins, M. Am. Soc. C. E.

He was a member of the American Association of Engineers, the Cornell Society of Civil Engineers, the Society for the Promotion of Engineering Education, the Southwestern Geological Society, and a Fellow of the Royal Society of Arts, London, England. He was also a member of the Protestant Episcopal Church, a Scottish Rite Mason, and a member of Ben Hur Temple, A. A. O. N. M. S.

Mr. Nagle devoted his life to his work in and for the State of Texas, and he was loved by all who knew him. The following tribute is quoted from the editorial column of the *Houston Chronicle* under date of April 8, 1928:

"Word that J. C. Nagle, former Dean of Engineering at A. & M., is dead, will bring regret to many thousands of men now active in the industrial life of Texas and the Southwest. This splendid engineer gave his talents for many years to training youths at the great Texas Mechanical College.

"His personal influence probably meant as much as the technical knowledge he imparted. He was a leader of men as well as a teacher. A State can never know what it owes to such men; their work lives in the lives of other men. Those other men know, however, and many of them will grieve to-day over the loss of their friend and mentor."

Mr. Nagle was elected a Member of the American Society of Civil Engineers on June 6, 1905.

GEORGE NAUMAN, M. Am. Soc. C. E.*

DIED SEPTEMBER 28, 1927.

George Nauman was born at Lancaster, Pa., on June 20, 1870. He was graduated from Lehigh University in 1890 with the degree of Civil Engineer.

In September, 1890, he entered the service of the Pennsylvania Railroad Company as a Rodman in the Engineering Department. He was successively promoted to the grades of Transitman in September, 1890, Assistant Engineer in February, 1899, and Assistant to the Chief Engineer in July, 1918, which position he held at the time of his death.

In his various capacities, Mr. Nauman had a wide experience in railroad location and construction. It was in the latter work that he found the most enjoyment during his very useful life. An executive, a worker, a co-ordinator, and a doer, he was most content when handling and expeditiously completing the many large construction projects that were under his supervision.

Among the larger projects constructed under his direction were the Delaware River Bridge from Philadelphia, Pa., to Delair, N. J.; the famous Rockville Bridge across the Susquehanna River, at Harrisburg, Pa.; the Shocks Mills Bridge across the Susquehanna River at Shocks Mills, Pa.; the four-tracking of the Northern Central Railroad from York Haven to Marysville, Pa.; the Enola Yards, near Marysville, Pa.; the Northumberland Yard, Northumberland, Pa.; the Montgomery Bridge at Montgomery, Pa.; the Allegheny Division improvements, including the Rockland and Kennerdell Tunnels; the Buffalo Division improvements between Oil City and Corry, Pa.; the Gardenville engine facilities, near Buffalo, N. Y.; the additional engine facilities at Pitcairn and Shire Oaks, Pa.; the Northside improvements, Pittsburgh, Pa., including the Northside track elevation; four-tracking, Pittsburgh to Sharpsburg, Pa.; and the new Sharpsburg Yard and Engine Terminal; the Beck's Run Viaduct; and the Streets Run Viaduct on the Monongahela Division, at Pittsburgh. In addition to these larger projects he planned and constructed many minor railroad improvements, including grade-crossing eliminations and branch-line extensions.

Mr. Nauman's executive superiors held him high in their esteem, recognizing his excellent judgment and abilities. He enjoyed the social side of life during his active career, making many friends who always appreciated his ready wit and repartee. His wise counsel and sympathetic nature won for him the affection of his subordinates who appreciated their contacts with such a fine outstanding character. The profession has sustained a real loss in the passing of Mr. Nauman, a foremost Construction Engineer.

He was a member of the Engineers Society of Pennsylvania, the Harrisburg Club, of Harrisburg, and the University Clubs of Philadelphia, Pa., and of Pittsburgh.

Mr. Nauman was elected a Member of the American Society of Civil Engineers on May 4, 1904.

* Memoir prepared by T. P. Watson, M. Am. Soc. C. E.

GEORGE ALBERT NOSKA, M. Am. Soc. C. E.*

DIED NOVEMBER 12, 1928.

George Albert Noska was born on April 5, 1873, in Philadelphia, Pa. He was educated in the public schools of that city and, later, attended the Philadelphia Manual Training School.

In 1891, Mr. Noska started work with the Pencoyd Bridge Company, with which he remained until 1897, when he entered the employ, as Steel Designer, of the late George B. Post, M. Am. Soc. C. E., in New York, N. Y. He held this position until 1899, with the exception of the time he spent in service during the Spanish-American War. He was Corporal of Company M, 71st New York Volunteer Infantry, and saw service in Cuba, at San Juan Hill, and in other engagements. Subsequently, he served in the New York National Guard and was commissioned First Lieutenant of the Depot Infantry, National Guard of New York, on August 2, 1916.

In 1899, Mr. Noska became associated with Francis Kimball, Architect, as Superintendent of general construction, with whom he remained until 1908, when he was employed in the offices of D. Everett Waid, Architect. In this position his time was largely taken up in passing on plans on which the Metropolitan Life Insurance Company proposed loans.

In May, 1917, when the United States entered the World War, Mr. Noska again joined the Army. He was enrolled at Madison Barracks, Sacketts Harbor, N. Y., as Student in the Cadet Engineer Corps and was assigned to the 78th Division. He was discharged from service on July 2, 1919, at Camp Dix, New Jersey, as Captain of the 303d Engineers, 78th Division. He served overseas from May, 1918, to June, 1919, and saw active service at Chateau-Thierry, Meuse-Argonne, and St. Mihiel, and was awarded two gold chevrons. After his discharge he was commissioned Major in the 1st Battalion, 303d Engineers, 78th Division, Officers Reserve Corps. He was intensely patriotic as his record shows and was 44 years old at the time of his enlistment in the World War. He refused to consider any but active service at the front and undoubtedly sowed the seed there which developed into the heart attack which caused his death.

Following his discharge in 1919, Mr. Noska became a member of the firm of Chris. J. Jeppessen, Incorporated, until 1923, when he entered into partnership with Mr. Charles L. Lange, under the firm name of Lange and Noska. He was conducting this business at the time of his death on November 12, 1928. During these latter associations involving work around New York City, he was engaged on the designs of the Canadian-Pacific Building, Saks Department Store (Fifth Avenue), the building at Broadway and 40th Street, the Williamsburgh Savings Bank, the Riverside Church, and innumerable other structures.

Mr. Noska was married, on October 9, 1905, to Margaret Peasley. He is survived by his widow and two daughters, Evelen (Noska) Douglass and Ruth Adelaide Noska.

Mr. Noska was elected a Member of the American Society of Civil Engineers on September 5, 1911.

* Memoir prepared by Charles L. Lange, Esq., New York, N. Y.

EDWARD JONES PEARSON, M. Am. Soc. C. E.*

DIED DECEMBER 7, 1928.

Edward Jones Pearson, the second eldest of a family of six children—three sons and three daughters—of Leonard and Lucy S. (James) Pearson, was born at Rockville, Ind., on October 4, 1863. His father followed railroading as his profession and held prominent positions as a railroad officer during his lifetime. The family was of English origin, John Pearson the earliest paternal American ancestor having come from England about 1630 to 1635, and having settled at Rowley, Mass.

After elementary preparation in the public schools of Chicago, Ill., Mr. Pearson entered Cornell University at Ithaca, N. Y., from which he was graduated with the degree of Civil Engineer in 1883. He was a member of the Delta Upsilon Fraternity.

He began his railroad work during the summer vacation of 1881, as a Rodman in the Engineering Department of the Missouri Pacific Railway Company, on the extension from Atchison, Kans., to Omaha, Nebr., and also on construction work in Indian Territory and in Texas. In the summer of 1882 he was engaged as Assistant Engineer in terminal construction work at Portland, Ore., and after graduating in 1883 was employed by the Northern Pacific Railway Company as Supervisor of Track, until 1884, when he was placed in charge of construction train service. From 1890 to 1892 he was Division Engineer on lines east of Livingston, Mont., and was then made Principal Assistant Engineer of the Wisconsin Central Railway Company, the Chicago and Calumet Transfer Company, and the Chicago Terminal (leased by the Northern Pacific Railway Company), in charge of terminal construction.

Up to this time Mr. Pearson's work had been engineering exclusively. He had always shown, however, a keen interest in, and the desire to be associated with, operation and in 1894 was appointed Superintendent of the Yellowstone Division. He served the Northern Pacific Railway Company as Superintendent of the Rocky Mountain and Pacific Divisions until 1902, when he was made Assistant General Superintendent. In 1903, he was appointed Acting Chief Engineer, and, in 1904, Chief Engineer.

Mr. Pearson continued in this work until December, 1905, when he took charge as Chief Engineer of the Pacific Coast Extension of the Chicago, Milwaukee and St. Paul Railway, a line through rugged country, 1 350 miles long, which he built in as many days. He organized and started the operation of this extension. In 1911, he was appointed First Vice-President of the Missouri Pacific and the St. Louis, Iron Mountain and Southern Railway Companies, in charge of operation, thus returning to the first railroad on which he had served as a Rodman thirty years before.

Mr. Pearson was associated with the Missouri Pacific Lines until April, 1915, when he left to become First Vice-President in charge of the Texas

* Memoir prepared by C. E. Smith, M. Am. Soc. C. E., and M. K. Dugan, Esq., New Haven, Conn.

and Pacific Railway. During his four years of service with the Missouri Pacific System, the physical condition and capacity of the property was greatly improved, its operation re-organized, and its earning power brought to a much higher level.

He remained with the Texas and Pacific Railway Company only about a year, that is, until March, 1916, when he was called to the service of the New York, New Haven, and Hartford Railroad Company, as Assistant to the President, and Vice-President. He became President of the Company on May 1, 1917, on the resignation of the late Howard Elliott, Affiliate, Am. Soc. C. E. During the period of Federal control, Mr. Pearson was Federal Manager of the railroad, following which he was again elected President.

When Mr. Pearson entered its service, the New Haven Railroad was ill-equipped for handling the large volume of freight business being offered. He remodeled the road from a combination of many inadequate separate yards and engine terminals, to a consolidated system with adequate terminals at central points. Heavy locomotives were provided, bridges strengthened, passing tracks lengthened to enable the handling of modern trains, and its operations were completely re-organized. Dividends, which had been suspended in 1913, were resumed shortly before his death.

It is worthy of note that, largely because of his caution in railroad operation, not one passenger was killed in a train accident on the New Haven Lines during twelve and one-half years of his administration. Mr. Pearson's service to the New Haven Railroad Company and to the territory it served is fitly expressed in the following quotation from the remarks of Dr. A. T. Hadley, Director of the New Haven Railroad, and President Emeritus of Yale University, at the funeral service:

"Never was a man faced with a more discouraging task than that which confronted this man in 1920. Only those who were at his side in those dark days can know the burdens which he was called upon to carry, and the superb way in which he bore them. Step by step, with infinite patience and undaunted courage, he brought order out of chaos in the operations and finances of the New Haven System and put it in a position to serve the community under present-day traffic conditions."

The outstanding traits of Mr. Pearson's character were his desire to avoid publicity, his modesty as to his accomplishments, and his kindly consideration and loyalty for his associates, particularly those junior in authority—his desire that they should be given every opportunity for advancement and full credit for good work well done.

In June, 1926, the degree of Master of Arts was conferred upon Mr. Pearson by Trinity College. He was a member of the American Railway Engineering Association, the Cornell Society of Civil Engineers, and the Connecticut Society of Civil Engineers; he was also a Director of the American Railway Association. He belonged to the Bankers, the Engineers, and the Metropolitan Clubs of New York, N. Y.; the Graduates and Quinnipiack Clubs, of New Haven, Conn.; the Metropolitan Club of Washington, D. C.; the New England and New York Railroad Clubs, and the Traffic Club of New England.

Mr. Pearson was married to Gertrude S. Simmons at Evanston, Ill., on June 7, 1899. His widow and a son, Harlow S. Pearson, survive him; also his mother, two sisters, and a brother, Raymond A. Pearson, President of the University of Maryland.

Mr. Pearson was elected a Member of the American Society of Civil Engineers on December 4, 1907.

WILLIAM GUNN PRICE, M. Am. Soc. C. E.*

DIED JULY 6, 1928.

William Gunn Price was born in Knoxville, Pa., on July 6, 1853. His father, William Price, was a physician of Welsh and English descent. His mother, Tryphene R. (Gunn) Price, was of Irish and Scotch ancestry. He gained his early education in the public schools of New York State, at Hartwick Seminary, and later, at Columbia University.

Mr. Price's engineering experiences began when he was eighteen years of age, in the vicinity of New York and, from 1879 to 1896, he was United States Assistant Engineer, in charge, successively, of surveys on the Mississippi River, the improvement of the Harbor of New Orleans, La., the rectification of the Red, Mississippi, and Atchafalaya Rivers, and other important works. He had charge of the construction of the two dams at the head of the Atchafalaya River, which prevented the Mississippi from flowing to the Gulf by that shorter route, in the disastrous flood of 1927. He also assisted in the first survey of the Mississippi, which previously had not been mapped accurately, and he took part in a similar survey of the Missouri River.

Mr. Price was the inventor of the electric current meter and the acoustic current meter for measuring the flow of water. Both these instruments are extensively used by engineers all over the world. At New Orleans, he designed a new and successful system of spur-dikes for protecting the banks of the river. He was also the originator and advocate of the plan for the improvement of the Lower Mississippi River, by the method which utilizes the forces of the river to dig the foundation for permanent channel-controlling works, which plan was successful as applied by him in New Orleans Harbor, the Atchafalaya River, and elsewhere. This method of construction is such that the force of the river can only sink the structure deeper in the sand while more material is being added on the top. All his inventions and improvements in engineering construction have been boldly original.

In 1896, Mr. Price became Chief Engineer for the Chicago City Railway Company, and produced the first railway track test car. This car, when run over the road at train speed, using five pens on a moving sheet of paper, recorded the location and amount of all defects in the track.

The same year he originated the momentum power brake for street cars, which was used on all the Chicago lines and by many other railways, until the advent of much larger cars necessitated the use of the air-brake. Mr. Price also invented an automatic fare box for street cars which prevented drivers from "knocking down" fares.

For a period of ten years, beginning with 1903, he was employed by the Standard Steel Car Company, of Pittsburgh, Pa., to design improvements in railway equipment. The Standard Motor Truck Company, a subsidiary Company, was incorporated especially to control the manufacture and sale of Mr. Price's inventions. During these ten years he placed with the Standard

* Memoir prepared from information on file at the Headquarters of the Society.

Motor Truck Company, on royalty, thirty-three patents of railway equipment, many of which are in extensive use in the United States and other countries.

In 1913 Mr. Price went to Yakima, Wash., and contributed to labor-saving devices in the fruit industry, such as, a fruit-sizing machine; an automatic elevator, which, in the warehouses, saves the work of several men; box-making machines; box presses; an improved gravity conveyor; a machine for sizing potatoes; and a box press for dried apples. Not the least interesting of his inventions, is an ingenious little vest-pocket pencil sharpener. His last invention was a hydraulic friction brake for automobiles.

Mr. Price had not been in the best of health for two and a half years prior to his death, having a leakage of the heart; but in the late spring and early summer of 1928 he was feeling so much improved that on June 20 he started with his wife on a trip to Central New York. They stopped en route at the Battle Creek Sanitarium for a physical examination. Receiving no benefit from a two weeks' stay at the institution, Mr. and Mrs. Price resumed their journey. Mr. Price's condition became serious, however, and at Detroit, Mich., he was taken from the train to the Fort Shelby Hotel, where he died the next day, Friday, July 6, 1928, the seventy-fifth anniversary of his birth.

He is survived by his wife, Mary Kelley Price, a native of Schenevus, N. Y., to whom he was married in Chicago, in 1879; one son, William Kelley Price, of Selah, Wash., and one grandson, William Griffin Price.

Mr. Price was a member of the American Association of Engineers, Professional Engineers, and the American Association for the Advancement of Science; he was also a Mason and belonged to the Scottish Rite and the Shrine.

Mr. Price was elected a Member of the American Society of Civil Engineers on April 3, 1895.

ARTHUR BREESE PROAL, Jr., M. Am. Soc. C. E.*

DIED AUGUST 22, 1922.

Arthur Breese Proal, Jr., was born on October 10, 1869, in New York, N. Y., the son of Arthur Breese Proal, a prominent banker and financier of New York. His mother was Sydney Mathiot, of Zanesville, Ohio, the daughter of the Hon. Josiah Mathiot.

Mr. Proal received his education at Stevens Preparatory School and was graduated from Pratt Institute in Brooklyn, N. Y. He began his first practical work in 1892 in the Electrical Testing Department of the Metropolitan Telephone Company, which later became the New York Telephone Company. He was engaged mostly in laying underground cable systems in New York City and assisted in the laying of the first cable under the Hudson River.

In December, 1892, he was engaged as Construction and Mechanical Engineer for the Newark Electric Light and Power Company of Newark, N. J. (now the Public Service Corporation of New Jersey). In this capacity, he had charge of the design and erection of steam and electric equipment, as well as the construction of several large power houses, together with detailed work for docks and intakes on the water-front.

In November, 1896, Mr. Proal was made Superintendent and Chief Engineer of the New Haven Street Railway Company, at New Haven, Conn., for which he had charge of the entire operation of the System and the extension of its service. During this period, he laid out and built a number of miles of interurban trolley line and acted as one of the pioneers in the construction and operation of Light House Park at the end of the Interurban Line.

In January, 1898, he became Superintendent and Chief Engineer of the New York and Staten Island Electric Company, which, in addition, controlled the Southfield Beach Railway Company. Mr. Proal built the latter road complete, including its entire power equipment. He was also Superintendent and Chief Engineer of the New York and New Jersey Ferry Company, the Richmond Light, Heat, and Power Company, and of several smaller companies. At this time he designed and supervised the construction and erection of the central power plant—7 000 h.p.—that furnishes all the lights and railway current in Richmond County, New York.

In January, 1901, Mr. Proal became associated with the Robins Conveying Belt Company, of New York, as Sales Engineer, with which company he remained until his death from pneumonia on August 22, 1922. He was ill but a few days.

Mr. Proal was the inventor of many devices used in the art and construction of conveyances and was particularly prominent in the planning and design of many of the important vessel-loading plants on the Atlantic Coast. He also developed and patented two types of bridge tramways for open storage of materials, which are in successful operation throughout the world to-day.

* Memoir prepared from information on file at the Headquarters of the Society.

These plants stand as worthy memorials to his remarkable character and ability.

Coupled with his exceptionally observant and perceptive nature, Mr. Proal had a very retentive memory and supplemented his extended engineering knowledge with the rare ability of being able to transmit this knowledge to others concisely and freely. His death was mourned by his associates not only because of his cheerful disposition, his great mechanical ingenuity, and his sales ability, but also because of his exceptional fund of knowledge and his willingness to impart it. He left a vacancy impossible to fill.

In October, 1906, Mr. Proal was married to Sara Francisco Harris, of Newark, N. J., who survives him.

Mr. Proal was elected a Member of the American Society of Civil Engineers on March 7, 1906.

HARLEY EDSON REEVES, M. Am. Soc. C. E.*

DIED JULY 28, 1928.

Harley Edson Reeves was born at Tampico, Ill., on March 21, 1874. He received his early education at Tampico, and afterward attended the University of Illinois where he received the degree of Bachelor of Science in Civil Engineering in 1895. While at college, he was known as one of the best football players of his time.

Mr. Reeves began his engineering career as a Sergeant and Lieutenant in the 3d Volunteer Engineers during the Spanish-American War, on topographical work, construction of temporary bridges, block-houses, field fortifications, and signal towers. His service in the Army extended from June 26, 1898, to May 17, 1899.

On October 1, 1899, he was appointed Rodman on the construction work of the Illinois and Mississippi Canal and was promoted, successively, as Inspector, on May 1, 1900; Junior Engineer on February 1, 1902; Assistant Engineer on September 1, 1911; and Associate Engineer on October 1, 1926, which position he held until his death.

Between 1900 and 1907 Mr. Reeves was in local charge of earthwork, pile-driving, construction of superstructures and concrete abutments for bridges, foundations, and concrete superstructures for aqueducts, locks, and lock foundations for the Illinois and Mississippi Canal. When the Canal was completed he was placed in local charge of the operation and care of the Western Section, and later he was in charge of a preliminary examination and survey of Rock River with a view to securing a channel from Sterling, Ill., to Janesville, Wis., and to ascertaining whether for the maintenance of navigation, storage reservoirs were necessary in compliance with the provisions of the River and Harbor Act of March 3, 1909.

He was in local charge of the construction of the Rock River Lock, at Rock Falls, Ill., completed in 1912, and of the Le Claire Canal Lock and Dam on the Mississippi River, at Le Claire, Iowa, completed in 1922. From 1922 until his death he was in local charge of a section of the Mississippi River Improvement from the mouth of the Wisconsin River to Moline, Ill.

Mr. Reeves was a capable, resourceful engineer and took great pride in carrying out the various works upon which he was engaged, from the preliminary plans to construction and operation. He was a friend of his fellow employees and was respected and beloved by all those who were privileged to know him.

He was married on November 28, 1901, to Helen Nowers, at Atkinson, Ill., by whom he is survived, with one son, James Edson, two brothers, Howell Reeves, of Albany, N. Y., and Rear-Admiral Joseph M. Reeves, U. S. Navy, and a sister, Miss Frances Reeves, of Schenectady, N. Y.

Mr. Reeves was elected a Member of the American Society of Civil Engineers on January 4, 1910.

* Memoir prepared by Richard Monroe, M. Am. Soc. C. E.

MORRIS JOHN RIGGS, M. Am. Soc. C. E.*

DIED FEBRUARY 7, 1926.

Morris John Riggs was born on his father's farm at Horton, Bremer County, Iowa, on January 14, 1862. His parents were William and Sophronia Riggs, early pioneers of that State. He was educated in the district schools of Bremer County, in the High School, at Waverly, Iowa, and in Iowa State College, at Ames, from which he was graduated in June, 1883, with the degree of Bachelor of Civil Engineering, having specialized in bridge engineering. In 1905, he obtained from his Alma Mater the degree of Civil Engineer.

Before entering college in 1880, Mr. Riggs had spent four years teaching in the district schools. He worked his way through college, milking cows and performing other services on the Agricultural Farm. He was also the first Business Manager of Athletics at Ames, and was known as the "Father of the Memorial Union", of which he was President for five years, later becoming Honorary President. It was largely due to his tireless efforts that more than $1 000 000 was subscribed to the Union. As an Alumnus he always retained great interest in his Alma Mater, making frequent visits to the college. From 1920 to 1924, he served as President of the Alumni Association.

Following his graduation Mr. Riggs was employed for about a year making county maps in Kansas. In the spring of 1885, he became Bridge Engineer for Mr. S. M. Hewett, Agent, at Minneapolis, Minn., for the Smith Bridge Company of Toledo, Ohio. In July, 1887, Mr. Riggs was appointed Draftsman, Designer, and Estimator for this Company and continued in that position until April, 1893, when he became Chief Engineer of the Toledo Bridge Company, the successor to the Smith Bridge Company. The Toledo Bridge Company, in turn, was succeeded by the American Bridge Company and in April, 1901, Mr. Riggs was appointed Manager of the Toledo Plant of this firm, in which capacity he served until his death. In connection with his duties he had charge of numerous important works, among which was the design and construction of the Toledo Plant of the American Bridge Company. During the many years he served these several bridge companies, he also acquired splendid experience in bridge engineering.

At the time of his death Mr. Riggs was President of the Toledo Section of the Society, of the Chamber of Commerce, of the Young Men's Christian Association, at Toledo, and also of the Ohio State Young Men's Christian Association. He had served the local organization as President for sixteen years and was very much interested in its work, serving also as a member of its National Council. He was a member of the Ashland Avenue Baptist Church and leader of its Men's Class. During the World War he was very active in the several Liberty Loan Campaigns. On one occasion (in 1910),

* Memoir prepared by Walter J. Sherman and George A. Taylor, Members, Am. Soc. C. E.

without solicitation he was nominated for Mayor of Toledo on the Progressive Ticket. Mr. Riggs was also a member of the Inverness Club of Toledo.

On October 11, 1893, Mr. Riggs was married to Alma M. Fassett, daughter of the late Elias Fassett, an early pioneer of Toledo.

On Thursday, February 4, 1926, Mr. Riggs complained of an earache while returning from a trip to Detroit, Mich. He became unconscious on Friday and remained so until his death on Sunday, February 7, 1926, the cause of which was announced as spinal meningitis. He is survived by his widow, Alma Fassett Riggs; a sister, Sarah M. Riggs, Head of the Department of History in the State Normal School, at Cedar Falls, Iowa; another sister, Mrs. Luderne Franklin, of Chicago, Ill.; and one brother, Marvin L. Riggs, who lives on the old 240-acre homestead at Horton, Iowa.

On February 9, 1926, memorial services were held at the Ashland Avenue Baptist Church, attended by delegations representing the Young Men's Christian Association, the Iowa State College, the Chamber of Commerce, the Rotary Club, and the American Bridge Company. Testimonials were given by officers of the Church and Sunday School and by members of the numerous delegations present.

At Rotary, silent prayers were offered and an appropriate tribute was delivered. At the time of Mr. Riggs' funeral the Chamber of Commerce was closed out of respect to the memory of one of its most valuable members, and the City Council declared in a resolution that he had been one of the City's outstanding civic leaders. His was a "life widely useful, strangely courageous, and always busy with a tenderness, persistency, and loyalty known to few men."

In the death of Morris John Riggs, the City of Toledo has lost an ideal citizen, the American Bridge Company, an efficient Managing Engineer, the Chamber of Commerce, an earnest worker for the interests of his adopted city, the engineers of Toledo a sympathetic friend, the Young Men's Christian Association of Toledo, a most liberal supporter, and the Church to which he belonged, an earnest Christian laborer.

Mr. Riggs was elected a Member of the American Society of Civil Engineers on December 26, 1899.

RUDOLF VIEDT ROSE, M. Am. Soc. C. E.*

DIED JANUARY 25, 1928.

Rudolf Viedt Rose, the only son of Adolf and Helene (Viedt) Rose, was born in Niagara Falls, N. Y., on April 27, 1876. He was of fine old German stock, both his parents having come from Germany. His father left his native town of Hameln, in the Province of Hanover, to come to America, and settled at Niagara Falls where he became an honored and successful business man.

Mr. Rose was educated in the public schools of Niagara Falls and the Central High School, of Buffalo, N. Y. His engineering education was obtained in Stevens Institute of Technology, at Hoboken, N. J., from which he was graduated in 1897 with the degree of Mechanical Engineer.

Immediately after graduation, Mr. Rose began his active work as Draftsman with a manufacturing company in Buffalo, but shortly thereafter on September 15, 1897, he returned to Niagara Falls to enter the Engineering Corps of the Niagara Falls Power Company. He remained with this Company for twenty-three years (until and following its consolidation with the Hydraulic Power Company on June 1, 1920), filling many positions from Draftsman to Construction Engineer in connection with the construction and operation of the now famous hydro-electric power development on the Niagara River. During his earlier employment with the Power Company, he spent a six months' leave in Germany, working with Allgemeine Electricitäts Gesellschaft at Berlin.

In 1920, Mr. Rose entered commercial life as President and Manager of the Consolidated Fuel and Supply Corporation, which he conducted for several years, dealing in coal, contractors' supplies, and building materials. His company was then merged with the Empire Builders' Supply Company in Niagara Falls, and he held the position of Engineer and Superintendent in this larger industry until his death.

He was intensely interested in his professional work and always active and energetic. His was a pleasing personality and he endeared himself to a large circle of friends.

In 1907, Mr. Rose was married to Joyce Grant, of Toronto, Ont., Canada, who, with their three children, Carl Grant, Rudolf William, and Joyce Helene, survives him.

He was a member of the Niagara Club, the Niagara Falls Country Club, and the Honorary Society of Tau Beta Pi. He was a Vestryman of St. Peter's Protestant Episcopal Church.

Mr. Rose was elected an Associate Member of the American Society of Civil Engineers on April 5, 1905, and a Member on February 28, 1911.

* Memoir prepared by Walter McCulloh, M. Am. Soc. C. E.

EDWARD EMMET SANDS, M. Am. Soc. C. E.*

DIED OCTOBER 27, 1923.

Edward Emmet Sands was born in Columbus, Ohio, on January 5, 1877, the son of Louis K. and Emily Sands. His early education was received in the grade and High Schools of Sparta, Wis., and his technical education at the University of Wisconsin, from which he was graduated with a degree of Bachelor of Science in Civil Engineering in 1900. This was followed by a Civil Engineer degree in 1906.

During the winters of 1900 and 1902, Mr. Sands served as Instructor in Civil Engineering at his Alma Mater; during the summers he acted as City Engineer of Sparta, designing and supervising water, sewer, and paving improvements. From 1902 to 1908 he acted as Engineer in the United States Reclamation Service and from 1908 to 1911, as Manager for the Upper Columbia Irrigation Company (part of 1911 and 1912 with the United States Reclamation Service). From April, 1912, to September, 1913, he served as Supervising Engineer for the Canadian Pacific Railway Company on irrigation and municipal projects in Calgary, Alberta, Canada.

In September, 1913, Mr. Sands became City Engineer of Houston, Tex., engaged in general city work and port development, the latter estimated at $3 500 000. During this service he designed and supervised the construction of the first "activated sludge" sewage disposal plant in America. During the World War he also acted as Supervising Engineer for Camp Logan and Consulting Engineer for the Ellington, Carstner, Brooks, and Kelly Aviation Fields.

From March, 1918, to March, 1920, he was engaged as Consulting Engineer for the Atlantic, Gulf, and Pacific Company, in New York, N. Y.; from March to September, 1920, he served as President of the Houston Construction Company, at Houston; and from September, 1920, to June, 1921, as Chief Engineer for the H. F. Friestedt Company, Chicago, Ill., and Houston.

In June, 1921, Mr. Sands entered private consultation practice in partnership with John B. Hawley, M. Am. Soc. C. E., of Fort Worth, Tex., with offices in Houston and Fort Worth. This firm designed and supervised the construction of water purification and sewage disposal works at Fort Worth, Breckenridge, Lubbock, and Sweetwater, Tex., and port improvements at Corpus Christi, Tex., totaling several million dollars, during 1921, 1922, and 1923.

In 1903, Mr. Sands was married to Isabel Garrison, of Denver, Colo. They had four children, Grace Isabel, Ruth Elizabeth, Emily Ann, and Edward Emmet, Jr.

At the zenith of his mental and professional activities, early in 1923, he was stricken with cerebral tumor; two operations failed to abate the malady, and while visiting his father at Milwaukee, Wis., the end came as he slept, on October 27, 1923.

* Memoir prepared by John B. Hawley, M. Am. Soc. C. E.

Edward Sands was known to his profession as an unusually erudite, skilful, practical, experienced, efficient, trustworthy engineer; to his engineering associates, as a sound scientist, able executive, and wise counsellor; to his social acquaintances and friends, as a genial, companionable, helpful comrade; to his family, as a considerate, loving husband and indulgent, doting father; to the writer, who will ever mourn his passing, as a dear friend and faithful coadjutor.

He was a Past-President of the Texas Section of the Society; Chairman of the Houston City Plan Commission, and a member of the following organizations: The University Club of Houston and the Houston Country Club; the Wisconsin Division of Sons of the American Revolution; Trinity Protestant Episcopal Church, of Houston; the Masonic Fraternity, having been a member of Hiram Lodge No. 5 of Madison, Wis.; and the Shrine, Arabia Temple, Houston.

Mr. Sands was elected a Member of the American Society of Civil Engineers on February 4, 1913.

GLENN MASON SCOFIELD, M. Am. Soc. C. E.*

DIED DECEMBER 21, 1926.

Glenn Mason Scofield was born in Hermon, N. Y., on May 3, 1873, the son of George Vincent Scofield and Emily Jane (Mason) Scofield, both of whom were of typical New England stock, their families having landed in Connecticut and worked their way northwestward to Northern New York. His father, who was born in Jefferson County, New York, on October 20, 1831, was a Hydraulic Engineer at a time when small power plants were the rule and hydraulic engineering was in its infancy. His mother was a woman of beautiful character, a school teacher before her marriage, and always a faithful worker in the church and social affairs of her town.

Mr. Scofield's early education was acquired in the Grammer School and High School at Hermon. He then entered the Engineering Department of Union College, at Schenectady, N. Y., from which he was graduated in 1897 with the degree of Civil Engineer.

His first professional experience was as Engineer with the Youngstown Bridge Company, after which he became Assistant Manager of the New York Office of the Company, and, finally, Manager of this office, handling work throughout the Eastern States.

In 1903, he and his brother, Edson Mason Scofield, M. Am. Soc. C. E., formed the Scofield Engineering Company of Philadelphia, Pa. A few years later the two brothers associated with themselves, Mr. Frank Daugherty, a Nationally known electrical engineer. In 1920, these three, with Mr. Ford J. Twaits, a well-known structural engineer, organized the Scofield Engineering Construction Company, of Los Angeles, Calif. These principals have handled engineering and construction work amounting to more than $300 000 000, of which the following are samples: A coaling station for the U. S. Navy, at Guantanamo, Cuba; the steel work for a coaling station for the U. S. Navy, at Manila Harbor, Philippine Islands; dry docks for the U. S. Navy, at League Island Navy Yard, Philadelphia, Pa., and at the Mare Island Navy Yard, San Francisco, Calif.; power stations for the Harwood Electric Company, at Hazleton, Pa., and for the City of Jacksonville, Fla.; concrete ships for the U. S. Shipping Board, during the World War, at San Diego, Calif.; the Pacific Mutual Life Insurance Building; the Biltmore Hotel; the Biltmore Theatre; the Hellman Bank Building; the Pershing Square Building; the Elks Club Building; the Barker Building; the Commercial Club; the Ebell Club; the Hospital of the Good Samaritan; the Title Insurance and Trust Building; and the Sears-Roebuck Buildings, all of Los Angeles, Calif.; the Cooper Arms Apartments, at Long Beach, Calif.; and the Pacific Southwest Trust and Savings Bank, at Pasadena, Calif., and many other similar examples of high-class construction.

Mr. Scofield had a wonderful personality and a great sense of humor, he was a good mixer and the life of the party wherever he might be. Although

* Memoir prepared by J. C. Davenport, Cost. Engr., Scofield Eng. Constr. Co., Los Angeles, Calif.

he was educated as an engineer, he early had concentrated his efforts on the selling end of the business in which his temperament and character made him specially effective. He never forgot a face and took such a personal interest in every new acquaintance that he could repeat the details of the meeting after years of separation.

Had he been spared longer, undoubtedly his works would have been greatly enlarged, although now they stand as an impressive monument to his talent and industry, and to that of his associates. Mr. Scofield is survived by his partners, Messrs. Edson Mason Scofield, Ford J. Twaits, and Frank Daugherty, who are carrying on the work he so ably helped to create. He was a member of the Union League of Philadelphia, and the University Club, the Elks Club, the Commercial Club, the Jonathan Club, and the Breakfast Club of Los Angeles.

He was married at Youngstown, Ohio, on September 14, 1899, to Rosa May Clevely. He died at Los Angeles on December 21, 1926, while in the prime of life. He left, in addition to his widow, a daughter, Marion Adele, and a son, Glenn Mason Scofield, Jr.

Mr. Scofield was elected a Member of the American Society of Civil Engineers on September 7, 1904.

ADDISON MOFFAT SCOTT, M. Am. Soc. C. E.*

DIED APRIL 27, 1927.

Addison Moffat Scott, the son of Chauncey L. and Margaret L. (Moffat) Scott, was born in Dryden, N. Y., on February 20, 1843. He was of mixed Scotch-Irish and English ancestry.

When he was about sixteen, and for several years thereafter, he attended the Ithaca Academy, at Ithaca, N. Y., directing his studies in large part to mathematics and surveying. While attending the Academy, he usually taught school during the winters.

In the spring of 1866 Mr. Scott made a trip to the Northwest and at St. Paul, Minn., joined a party then organizing under Gen. G. K. Warren, U. S. Corps of Engineers, for a survey of the Upper Mississippi River. On the disbanding of this party at St. Louis, Mo., late in 1866, Mr. Scott returned home and renewed his studies at the Ithaca Academy. While he was attending school in the fall of 1867, General Warren offered him a position as Assistant Engineer on the improvement of the Upper Mississippi, which was then in progress. He accepted this position with much hesitation, as it interfered with his plan of study, which included a course at college. He remained on this work for several years, having been employed the latter three years of the time, as Assistant Engineer on the construction of a large railroad and highway bridge built by the United States Government across the Mississippi River, between Rock Island, Ill., and Davenport, Iowa. He was active in this work from the first survey for the location of the bridge until its completion in the spring of 1873. On the completion of the bridge, General Warren, who had been transferred to the East with headquarters at Newport, R. I., tendered Mr. Scott a position under him there. This would have been accepted but for another, and what appeared a more promising, offer made about the same time.

In that year, 1873, the United States Government undertook the improvement of the Great Kanawha River, which was in the Ohio River Department, then in charge of the late Col. William E. Merrill, U. S. Corps of Engineers, M. Am. Soc. C. E., with headquarters at Cincinnati, Ohio. Mr. Scott was placed in local charge of this work, residing at Charleston, W. Va. Throughout the several changes that took place in the organization of the river improvement force, Mr. Scott was continuously retained in his position as Chief Assistant or Resident Engineer. During his long service there, he devoted himself with persevering zeal and energy to the duties of his professional position, mastering thoroughly the theory and practical details of the Chanoine movable dams which the Government adopted in the improvement of the Great Kanawha River. The first movable dams in America were built in this river.

Mr. Scott remained in local charge of the Great Kanawha River Improvement until the fall of 1901, when he resigned, establishing a record of continuous service of more than 28 years on that work and of 34 years as Civil Engineer in the United States Corps of Engineers. A reference to his pro-

* Memoir prepared by C. P. Fortney, M. Am. Soc. C. E.

fessional service should not omit the following tribute paid to him by a distinguished Engineer Officer, the late Gen. William P. Craighill, Past-President, Am. Soc. C. E., under whom he served for 21 years, a period embracing the beginning and nearly the entire completion of the lock and dam system of the Great Kanawha. General Craighill resigned from command of the Baltimore Division of River and Harbor Works, which included the Great Kanawha Improvement, in 1895, when promoted to the highest office in his Corps, the Chief of Engineers of the Army. In an article on the Kanawha Improvement,* General Craighill stated:

"I desire also to call attention in the most emphatic way to another member of the American Society of Civil Engineers, who has for more than twenty-five years been connected with this important improvement which has been most successful from a commercial as well as an engineering standpoint. I refer to Mr. A. M. Scott, who was Principal Engineer on this work when I took charge of it for the United States in 1874; he has remained on it up to the present time when it is completed. The conduct of the work has been most economical, wise and excellent in every way. Mr. Scott has exhibited an unusual degree of skill as a designing and constructing engineer in the management of the improvement and in dealing with the many perplexing problems which have presented themselves for solution in its progress. To him is due, more than to any other person, the success of this work. I say what I know, and it gives sincere pleasure thus to bear record to the merit of a most faithful and deserving man."

To all his subordinates Mr. Scott was always courteous and kind, winning their respect and esteem, while he himself possessed the unbounded confidence and kind regards of superior officers. In his profession he achieved an enviable reputation for capacity, skill, and executive ability. In his general intercourse with the public Mr. Scott was a modest, unassuming, courteous gentleman. He was accurate, prompt, and thoroughly reliable in all business engagements and transactions.

Mr. Scott continued to make Charleston his home until the time of his death. At one time he was Vice-President of the Charleston Chamber of Commerce; he was a Mason of long standing—Past High Priest of Royal Arch Masons and Past Eminent Commander of Knights Templar of the Charleston Commandery. He was long identified with the Kanawha Presbyterian Church of Charleston and was a member of its Board of Trustees. Although never prominent in politics, Mr. Scott took a real interest in all public matters. He was a life-long Democrat.

He was married in 1907 to Florence L. Jeroloman, a daughter of Judge John Jeroloman, of New York City, who survives him.

Mr. Scott was elected a Member of the American Society of Civil Engineers on June 2, 1886.

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* *Engineering Record*, June 17, 1899.

JOHN HENRY STAATS, M. Am. Soc. C. E.*

DIED JULY 6, 1928.

John Henry Staats, the son of John Henry Staats and Jennie (Mapes) Staats, was born in New York, N. Y., on November 18, 1855. His ancestors on his father's side were among the first families who settled in Albany, N. Y., having come from Holland in 1629. On his maternal side he was descended from the Mapes family who came to the United States before the Revolutionary War, and made its home in Coxsackie, N. Y.

Mr. Staats received his education in the public schools of New York City, and at City College. In 1872, he entered the office of Gen. Charles K. Graham to take up the study and practice of civil engineering.

In 1876 he was employed in the Department of Docks of New York City under General Graham, who had succeeded Gen. George B. McClellan as Chief Engineer of that Department. Mr. Staats served in the Department until 1882, at which time he had risen to the position of Assistant Engineer of Construction.

From 1882 to 1884 he was connected with the firm of Ross and Sanford, of Jersey City, N. J., which was engaged in river and harbor work in New York City and vicinity.

In 1888, with his brother, the late Robert Parker Staats, M. Am. Soc. C. E., he formed the partnership of R. P. and J. H. Staats which, later, was incorporated, and of which Mr. Staats served as Vice-President. During the time of its existence this firm constructed many notable works in New York Harbor, among which might be mentioned the North German Lloyd Docks in Hoboken, N. J., which were afterward used for the embarkation of troops during the World War, the Holland-American Line Docks, also in Hoboken, and three of the so-called Chelsea Piers in New York City below West Twenty-third Street. Mr. Staats was considered an expert on harbor work and dock construction and was retained by several of the transatlantic steamship companies in a consulting capacity.

In 1915, the Company was sold to the Snare and Triest Company of New York and Mr. Staats became Vice-President and Consulting Engineer of the latter firm. This position he held until his retirement from business in 1917.

He died suddenly at York Harbor, Me., on July 6, 1928, and is survived by his widow, Edythe Tillinghast Staats, and two daughters, Mildred Staats Nelson and Fredericka Staats Tuttle. He was a member of the Union League Club of New York, the Holland Society, and the Sons of the Revolution.

Mr. Staats was elected a Member of the American Society of Civil Engineers on March 5, 1884.

* Memoir prepared by Gerald Staats Rinehart, Assoc. M. Am. Soc. C. E.

CADY STALEY, M. Am. Soc. C. E.*

DIED JUNE 27, 1928.

Cady Staley was born at Scotch Bush, N. Y., on December 12, 1840, the son of Harmanus and Evaline (Darrow) Staley. He worked on his father's farm and attended a district school and Jonesville Academy until he was sixteen years old, when he entered the Classical Institute, in Schenectady, N. Y. He went from there to Union College, where he took the course in Civil Engineering under Professor Gillespie. He was graduated with honors in 1865 with the degree of Bachelor of Arts, and, in 1866, he was given the degree of Civil Engineer.

As soon as Mr. Staley was graduated from college, he crossed the plains with a wagon train as the driver of an ox team, and after part of a year spent in prospecting for gold, he was employed by the Central Pacific Railway Company as a Civil Engineer. As such he was engaged on tunnel construction during 1866 and 1867, after which he returned to Union College as an Instructor in Civil Engineering. In 1868, on the death of Professor Gillespie, under whom he had served, he was made Professor of Civil Engineering.

Professor Staley, in 1876, became Dean of the Faculty, which position he held until 1886 when he was called to Case School of Applied Science, Cleveland, Ohio, as Professor of Civil Engineering and President of the Faculty.

As a result of his years of experience at Union College, Dr. Staley brought to Case well-defined ideas as to the organization and administration of an Engineering School. Requiring that he have full charge of the work done in the school, and ever willing to assume responsibility therefor, he, nevertheless, allowed the head of each department full power in the administration of that department, looking to him for results only. The success of Case School has proved the soundness of Dr. Staley's judgment, and what it is to-day is a monument to him as an organizer and administrator.

During the time that he taught at Union College and during the first years at Case School of Applied Science he was active in the practice of his profession, devoting his time to Sanitary Engineering. Dr. Staley early recognized the merits of the separate system of sewerage, and was one of the pioneers in promoting its adoption. In this work he had associated with him, George S. Pierson, M. Am. Soc. C. E., and together they designed and superintended the construction of separate systems of sewerage in West Troy, and Schenectady, N. Y., and in Dayton, Ohio.

Due to the lack of suitable texts on stresses in framed structures, Dr. Staley privately printed, during his early years at Union, the following booklets: "Notes on Bridge Engineering" (1875); "Strength of Materials and Stability of Structures" (1876); and "Elements of Truss Bridges" (1878). During this time he also revised and republished Gillespie's "Surveying" and "Roads and Railroads", and he, together with Mr. Pierson, published "The Separate System of Sewerage". After his retirement in 1902, he edited "The Teaching of Jesus".

* Memoir prepared by Frank H. Neff, M. Am. Soc. C. E.

In 1886, Union College conferred on him the degree of Doctor of Philosophy, and, in 1888, he received the degree of Doctor of Laws from both Union College and Ohio Wesleyan University. In 1920, Union College conferred on him the degree of Doctor of Science.

Dr. Staley was married at Waterford, N. Y., on December 29, 1869, to Kate Holcomb, daughter of Elvin and Philetta (Hall) Holcomb.

After coming to Case School Dr. and Mrs. Staley traveled abroad a great deal during their summer vacations. He brought back from these trips a wealth of architectural material which he gave to his classes in a series of illustrated lectures. These lectures were also open to the general public and were largely attended.

Dr. Staley retired in 1902, in order that he and Mrs. Staley might have ample leisure to travel, for which both had a great liking. After a trip around the world, they made their home either in Florida, or in Amsterdam, N. Y., where he had purchased a farm at Minaville, near the place where he was born. They continued, however, in their foreign travels.

Dr. Staley was a great lover of the horse, and during his earlier years at Case he planned to retire upon a stock farm which he was starting in the West. He had imported Percheron stock from France, and looked forward with pleasant anticipation to the raising of heavy draft horses. However, a series of bad years caused him to abandon this plan and dispose of his stock and farm.

As a teacher, Dr. Staley had the faculty of presenting the principles of the comparatively simple science that Civil Engineering was in his early teaching days, in a way that fixed them in the minds of his students. In his duties as President, he ever had the welfare of the individual student uppermost in his heart. Whenever the application of a rule, well-designed for the good of the student body, might injure the future of an individual boy, he never hesitated to set it aside; and many a student of slow development, or whose preparation had been poor, owed his opportunity to start an engineering education, or his chance to make good after having started, to this quality of Dr. Staley.

Although retaining the title of Professor of Civil Engineering at Case for a number of years, Dr. Staley early turned over the teaching to those who were to succeed him, so that comparatively few had the privilege of working under him in engineering classes. Those who did, cherish the memory of the intimate personal relationship of the classroom, his lack of formality, and his sympathetic attitude toward their work. Although few had the classroom contact, there is no former student, nor member of the Faculty, who was at Case during the sixteen years that Dr. Staley directed its affairs, who does not hold his memory in highest esteem.

Dr. Staley was elected a Member of the American Society of Civil Engineers on March 17, 1869.

JOHN TRUESDALE STEWART, M. Am. Soc. C. E.*

DIED JUNE 9, 1928.

John Truesdale Stewart was born at Loda, Ill., on January 13, 1868, the son of William R. and Nancy (Barr) Stewart. He was of Scotch-Irish descent, his ancestors having emigrated to the United States before the Revolutionary War. One great-grandfather served in the War of the Revolution and another in the War of 1812.

Mr. Stewart was graduated from Grand Prairie Seminary, Onarga, Ill., in 1888. In 1893, he was awarded the degree of Bachelor of Science from the University of Illinois, Urbana, Ill., and, in 1909, that of Civil Engineer from the same University.

While Mr. Stewart was known principally as an authority on agricultural drainage, his activities took him into other engineering fields in various parts of the country. Beginning his engineering career as Rodman on land drainage work in 1890, he spent the years 1891 to 1892 on railroad work. From 1893 to 1897 he was engaged in general engineering practice in Illinois. During the following two years he was occupied in both drainage and railroad work, holding several positions during this period. In 1899, Mr. Stewart was appointed Field Assistant with the United States Geological Survey, a position which he held until 1904, when he became Drainage Engineer, in the United States Department of Agriculture, on drainage and irrigation work in South Dakota, North Dakota, Minnesota, Arkansas, and Florida.

In 1908 he was made Professor and Chief of the Division of Agricultural Engineering, at the University of Minnesota, Minneapolis, Minn. At this time he conducted research work on the drainage and development of peat lands, the durability of drain tile, and the efficiency and maintenance of open ditches; he also prepared as sole or co-author numerous papers, several of which have been published by the United States Department of Agriculture. Among these are: "Drainage of the Eastern Parts of Cass, Traill, Grand Forks, Walsh, and Pembina Counties, North Dakota";† "Farm Drainage, Minnesota Extension";‡ "Installation of an Experimental Drainage System, Minnesota Experiment Station";§ "Review of Drainage Legislation";‖ and numerous other papers on land drainage. In addition, he was senior author of "Engineering on the Farm".

Imbued with a strong sense of loyalty and patriotism, Professor Stewart was active in the military service of the United States. In 1893, he was made Brevet Captain in the Illinois National Guard. With the outbreak of the Spanish-American War, he organized a volunteer company of which he was elected Captain. He became a Major in the Engineers Reserve Corps in

* Memoir prepared by the following Committee of the Northwestern Section: J. A. Childs, *Chairman*, Adolph F. Meyer, and E. V. Willard, Members, Am. Soc. C. E.

† *Bulletin 189*, Office of Experiment Stations, U. S. Dept. of Agriculture.

‡ *Bulletin 13*, Office of Experiment Stations, U. S. Dept. of Agriculture.

§ *Bulletin 110*, Office of Experiment Stations, U. S. Dept. of Agriculture.

‖ New York State Library, 1905-08.

1917, and served as Lieutenant-Colonel of Engineers in 1918. Although much against his desire he was not permitted to serve overseas, he distinguished himself so as to receive honorable mention from his superior officers. He held a commission as Colonel in the United States Engineers Reserve Corps at the time of his death.

Following the close of the World War, Colonel Stewart was connected with the Portland Cement Association. In 1922, he resumed his consulting practice as a recognized authority on drainage and wet land development.

Always courteous, friendly, and interested, Colonel Stewart had many friends, both in and out of the engineering fraternity. He was a student of natural history, always fond of outdoor life, and he especially enjoyed hunting and fishing.

He was a member of the Society of American Military Engineers, the Western Society of Engineers, the American Society of Agricultural Engineers, the Engineers' Society of St. Paul, the American Legion, the Sons of the Revolution, the Society of the War of 1812; and Tau Beta Pi, Sigma Xi, Alpha Zeta, and Gamma Sigma Delta Fraternities.

On January 1, 1900, he was married to Ida Belle Wilson who survives him. He died June 9, 1928, and was buried in Arlington National Cemetery, with full military honors.

Colonel Stewart was elected an Associate Member of the American Society of Civil Engineers on September 6, 1905, and a Member on October 10, 1916.

KIRBY LEE STRICKLAND, M. Am. Soc. C. E.*

DIED DECEMBER 10, 1927.

Kirby Lee Strickland, the son of Thomas and Frances (Sewell) Strickland, was born on November 12, 1871, in Coweta County, Georgia. He was educated at the North Georgia Agricultural College, a branch of the University of Georgia, where he was in attendance three and one-half years.

In July, 1896, Mr. Strickland entered the employ of the Edgemoor Bridge Works, as Timekeeper and Assistant Engineer on the erection of steel structures, with which Company he remained until 1901, at which time the American Bridge Company was organized. The Edgemoor Bridge Works was one of the various companies which were amalgamated into the American Bridge Company, and on account of his field experience Mr. Strickland was appointed to inaugurate an accounting system for the Erecting Department of the newly organized Company. He was subsequently transferred to field duties, where he remained until late in 1901, when he entered the General Office at Pencoyd, Pa., as Assistant to Manager of the Erecting Department.

In April, 1904, Mr. Strickland was appointed Assistant to the Manager of the Eastern Division Erecting Department, with offices in Philadelphia, Pa., in which capacity he was employed until July, 1915. During this time he supervised the erection of steel structures of every description, including the Baltimore and Ohio and the Pennsylvania Railroad Bridges over the Susquehanna River at Havre de Grace, Md., and numerous bridges on the New York, New Haven and Hartford, the Philadelphia and Reading, and Southern, Railroads.

In July, 1915, he was appointed Division Erecting Manager of the American Bridge Company, at Chicago, Ill., which position he held until the time of his death. During this period he was in charge of the erection of many large and important structures, a few of which are the Chicago, Burlington and Quincy Railway Bridge across the Missouri River, at Kansas City, Mo.; the Union Pacific Bridge across the Missouri River, at Omaha, Nebr.; the St. Joseph and Grand Island Bridge across the Missouri River, at St. Joseph, Mo.; the Bismarck and Mandan Highway Bridge, over the Missouri River at Bismarck, N. Dak.; the Atchison, Topeka and Santa Fe Railway Bridge across the Mississippi River, at Fort Madison, Iowa; the large steel ore docks at Duluth and Two Harbors, Minn.; the Illinois Steel Company's open hearth buildings, at Gary, Ind., and South Chicago, Ill.; and many other erection projects for the United States Steel Corporation. During the World War Mr. Strickland was active in the erection of various projects for supplying the United States Government with war materials, notably the large powder plant for the du Pont de Nemours Company, at Nashville, Tenn.

The last, and one of the most important, structures erected under Mr. Strickland's supervision was the large highway bridge across the Carquinez Straits of San Francisco Bay, which was completed during the summer of

* Memoir prepared by the following Committee of the Illinois Section: Messrs. Albert F. Reichmann and O. F. Dalstrom, Members, Am. Soc. C. E. and the late A. F. Robinson, M. Am. Soc. C. E.

1927. It was while he was on a trip in connection with the completion of this work that he was taken sick with what proved to be his final illness.

Mr. Strickland was a man of pleasing personality with numerous friends and admirers, who knew him also as a man of unusual resourcefulness resulting from his broad experience. He was an interesting narrator of experiences and anecdotes, and took keen delight in relating stories pertaining to his boyhood days in the "Old South." He was held in high esteem by all his associates throughout his long term of service with the American Bridge Company, and he numbered among his friends many men prominent in the Engineering Profession throughout the United States. He was a member of the Western Society of Engineers and the Engineers' Club of Chicago.

Mr. Strickland had made his home in Evanston, Ill., a suburb of Chicago, since 1915. On March 16, 1907, he was married to Charlotte Barney, of Pottstown, Pa., who died in 1921. Two daughters by this marriage, Eleanor and Elizabeth, survive him. On November 17, 1923, he was married to Amy Louise Halvorsen, of Chicago, who, with a son, Kirby Lee, survives him. Mr. Strickland is also survived by his mother, Mrs. Ellen Byram, who has remained at the ancestral home in Georgia.

Mr. Strickland was elected a Member of the American Society of Civil Engineers on October 15, 1923.

PAUL BERTRAM TALLMAN, M. Am. Soc. C. E.*

DIED JULY 29, 1926.

Paul Bertram Tallman was born at East Orange, N. J., on January 24, 1884. He was the son of the late Stephen Sands Tallman and Martha Broadmeadow Tallman. His great-grandfather, Simeon Broadmeadow, came to the United States from England in 1825 and by a Special Act of Congress he was made a citizen. He was one of the early promoters of the manufacture of steel and gas in this country, besides being an inventor of National fame.

Mr. Tallman's early life was spent in East Orange, where he fitted himself for college. On account of ill-health, he was forced to abandon his course, but from this time forward, he became a student and completed his education as an engineer even more thoroughly than college curricula require. His daily work in engineering was followed with careful study of the related problems during his hours of leisure. His greatest recreation was the complete and successful study and solution of these problems and the systematic recording of his findings.

His practical engineering experience began during the summer of 1902, with the Clearfield Bituminous Coal Company at Rossiter, Pa. In January, 1903, he entered the service of the New York Central and Hudson River Railroad Company, in the Terminal Engineer's Office, where he served, successively, as Chainman, Rodman, Instrumentman, and Draftsman on work in connection with the Grand Central Terminal improvements. In 1906, he resigned his position with the Railroad Company and accepted that of Engineer and General Superintendent with Bunn and Nase, General Contractors, of New York, N. Y.

From 1909, until his death, Mr. Tallman was in the employ of the firm of Warren and Wetmore, Architects, of New York City. While in this position, his work was of a most varied character and comprised largely the design and installation of mechanical equipment and plants, and the supervision of the construction of numerous buildings, principal among which were the Union Station, Houston, Tex.; the Broadmoor Hotel, Colorado Springs, Colo.; the Ritz-Carlton Hotel, Atlantic City, N. J.; and the Plaza Addition, the Heckscher Building, Aeolian Hall, and the Goelet Building, New York City.

In 1923, when a new department was organized by the firm for conducting architectural and engineering supervision for loaning institutions, Mr. Tallman was selected as its head. Here, again, his records and studies were of great assistance in organizing and conducting the work. Notable among the buildings under supervision by this Department, were the Cooper Department Store Building, Los Angeles, Calif.; the Michigan Theatre and Office Building, Detroit, Mich.; the Penn Athletic Club, Philadelphia, Pa.; and 200 Madison Avenue, and the new Paramount Theatre and Office Building, New York City.

* Memoir prepared by P. M. Corry, M. Am. Soc. C. E.

The high esteem in which Mr. Tallman was held by his fellow members of the Building Committee for the new Paramount Theatre and Office Building, is shown in the following resolutions:

"*Whereas,* on the 29th day of July, 1926, there departed this life in Rockville Center, Long Island,

PAUL B. TALLMAN

one of the Members of the Building Committee of The Paramount Building, New York City. At a meeting of the said Committee duly held on the 2nd day of September, 1926, it was

"*Resolved,* that out of respect to the memory of the deceased, suitable action should be taken to mark the high regard in which he was held at all times as a gentleman and engineer of ability and wise counsel; and

"*Resolved,* that we deeply deplore his death and extend to his widow and son our deepest sympathy in the hour of their great grief; and

"*Resolved,* that a copy of these resolutions be forwarded to his family."

On October 21, 1909, Mr. Tallman was married to Edna Davey, of East Orange, who, with a son, Stephen, survives him.

Mr. Tallman was of a highly sociable nature and enjoyed the company of his many friends. He was an untiring worker and never spared himself when his time and energies were needed by his friends or business associates. In business dealings he never took advantage of another's mistakes. He was instinctively honest and fair. His oldest friends were his best friends.

He took an active interest in civic and religious affairs. He was a life-long member of the Presbyterian Church—the last four years a devoted member of the Rockville Centre Presbyterian Church. He was President of the Board of Trustees and also very active in the Men's Club and in the Sunday School.

In 1922, he was elected a Fellow in Perpetuity by the Metropolitan Museum of Art, New York City. He was also a member of the American Railway Association.

Mr. Tallman was elected a Junior of the American Society of Civil Engineers on October 3, 1905, an Associate Member on September 10, 1910, and a Member on May 28, 1923.

EUGENE AUGUSTUS HOFFMAN TAYS, M. Am. Soc. C. E.*

DIED JULY 22, 1928.

Eugene Augustus Hoffman Tays was born at West Point, N. Y., on October 24, 1861, the son of the Rev. Joseph Wilkin Tays and Jennie (Crowell) Tays. At the time of his birth his father was teaching mathematics at the United States Military Academy. His ancestry on his father's side was Scotch-Irish, whereas on his mother's side it was English.

Mr. Tays received his early training in the Maysville, Ky., Institute and the Burlington, N. J., Military College. In the fall of 1880 he matriculated at Union College, Schenectady, N. Y., where he studied Civil Engineering, but owing to illness he was compelled to leave college at the close of his second year. While he was at Union he joined the Delta Upsilon Fraternity, and took several prizes in his military course for tactics and marksmanship.

Mr. Tays' first work in his chosen field was as Rodman, Leveler, and Topographer for the Mexican Central Railroad Company, on which road he later served as Division Engineer of Construction (1882-1883). In 1884 he was an Engineer on the Topolobampo and Pacific Railroad, and in 1885 he was engaged as Engineer in charge of the irrigation of a vast tract of land in the District of Fuerte, Sinaloa, Mexico.

In 1886, Mr. Tays went into private practice and was employed successively by the Mexican-American Construction Company of Sinaloa, the Sinaloa and Chihuahua Railroad Company, and the Mexican Western Railroad Company. In 1891 and 1892, he was Chief Engineer of the Topolobampo Canal, and, in 1893, Engineer and Superintendent of the San Javier Silver Mines of Sinaloa.

At this time Mr. Tays left the field of Civil Engineering for that of Mining Engineering, in which he became an expert, called on by the United States Government in expert testimony. He spent twenty or more years in this field of endeavor. During this time he was connected with the Palmarejo Mining Company of Palmarejo, Chihuahua, Mexico; the Mexican Mineral Railroad Company; the San Agustin Mining Company of San Rafael, Chihuahua; the Anglo-Mexican Mining Company of San José de Gracia, Sinaloa; the Compañía Minera de Santo Tomás y Anexas at San José de Gracia; the Maconí Mines, Maconí, Querétaro, Mexico; the United Mining Company of New York; the United Sugar Companies, Los Mochis, Sinaloa, Mexico; and the Compañía Minera Occidental Mexicana of South America, at San Blas, Mexico.

Because of the unsettled conditions in mining operations in Mexico in 1924, Mr. Tays turned to agriculture, and organized the Liga Agrícola Occidental Mexicana, of which he was the Managing Director. He was engaged in this most successful venture at the time of his death.

Mr. Tays was a constant contributor to scientific publications on mining, milling, metallurgy, geology, etc. One of his contributions was on "The

* Memoir prepared by the Rev. W. N. P. Dailey, New York, N. Y.

Economics of Mining". His studies as published in the *Mining World, Engineering and Mining Journal, Mining and Scientific Press, Western Mining World,* and the *Transactions* of the American Institute of Mining and Metallurgical Engineers, are notable contributions. He was affiliated with the following technical, scientific, and social organizations: The American Institute of Mining and Metallurgical Engineers; the United States Public Reserve (during the World War); the American Geographical Society; the Santa Cruz Club, of Nogales, Ariz.; the Society of Arts, London, England; and the American Association for the Advancement of Science.

He was married, in 1885, to Rosaura Vega, daughter of the Governor of the District of Fuerte, Sinaloa, Mexico. They had eight children, José W., Eloisa, Eugene, Clement I, Linda, George, Alexander, and Clement II, six of whom, with his widow, survive him.

Mr. Tays seems to have had at least three strong ties with which he bound himself up in life—his home and family, his profession and service, and his college and class. An Episcopalian in religion, he nevertheless held himself as a servant of all men wherever he might be of use, and through the years of his life felt that he was worshiping his Creator in a most accepted way by even the lowliest types of service in which he found himself of aid to his fellow men. A workman of distinction and varied acceptance in his chosen profession, he also possessed a high-mindedness of soul and fineness of character—attributes not as common as they ought to be among men of the present time. Even in the day of his departure, his was the vision splendid of better service and greater usefulness in the years to come—an indomitable spirit of optimism, his home and work holding the center of his affections and dreams.

Mr. Tays was elected a Member of the American Society of Civil Engineers on April 6, 1898.

STURGIS HOOPER THORNDIKE, M. Am. Soc. C. E.*

DIED FEBRUARY 16, 1928.

Sturgis Hooper Thorndike was born on June 11, 1868, at Beverly, Mass., the son of Samuel Lothrop and Anna Lamb (Wells) Thorndike. Of English ancestry, his earliest known paternal ancestor was William Thorndike, who died at Little Carlton, Lincolnshire, England, in 1539. His earliest paternal ancestor in America was John Thorndike, who came from Little Carlton to Ipswich, Mass., in 1633, but who returned to England, leaving in America his son, Paul, who settled in Beverly, Mass. His earliest known maternal ancestor was Harold de Vallibur (*i. e.*, Welles), who lived in Cumberland County, England, early in the Twelfth Century; his descendant, Hugh Wells, came from Essex County, England, to Watertown, Mass., in 1630, later going to Hartford, Conn., and then to Hadley, Mass.

Sturgis Thorndike's boyhood was spent largely in Boston and in Cambridge, Mass. After preparation at a private school, he entered Harvard College, where he received his Bachelor of Arts degree in 1890. Later, having decided to follow an engineering career, he entered the Massachusetts Institute of Technology, from which he was graduated with the degree of Bachelor of Science in Civil Engineering in 1895.

Entering the office of the City Engineer of Boston in 1895, at the beginning of a period of notable activity in city bridge construction, he gained during the next eighteen years a broad experience in the municipal engineering service of that city, primarily in its bridge building undertakings. In 1906 he was appointed to the position of Assistant Engineer in charge of bridge design. In 1911, on the consolidation of the Engineering, Street, and Water Departments into the Department of Public Works, he became the Designing Engineer of the Bridge and Ferry Division of this, the largest of Boston's municipal departments. In this position he was in charge of the design of bridges and of other municipal structures.

Among the more important Boston bridges with which Mr. Thorndike had engineering connection, may be mentioned the Charlestown, Summer Street, Northern Avenue, Chelsea Street, Chelsea North, Meridian Street, Broadway, and Atlantic Avenue (or Cove Street) Bridges, all over tide-water and each having an important draw-span; the Cambridge Bridge (now known as the Longfellow Bridge), across the Charles River between Boston and Cambridge, the most notable of the bridges of Greater Boston; and many important bridges on land, spanning railroad locations.

During this period, under leave of absence from the City Engineer during term time, Mr. Thorndike served as Instructor at the Massachusetts Institute of Technology in the scholastic years of 1904-05 and 1905-06, teaching hydraulics, stereotomy, bridge design, and surveying.

* Memoir prepared by Frederic H. Fay and Charles M. Spofford, Members, Am. Soc. C. E.

In October, 1911, he resigned from the position of Designing Engineer of the Bridge and Ferry Division of the City of Boston, to engage for a time in private engineering practice.

On July 1, 1914, Mr. Thorndike joined with Frederic H. Fay, M. Am. Soc. C. E., former head of the Boston Bridge and Ferry Division of the City of Boston, and with Charles M. Spofford, M. Am. Soc. C. E., the head of the Civil Engineering Department of the Massachusetts Institute of Technology, in forming the engineering firm of Fay, Spofford, and Thorndike. In this organization he was an active partner until his death.

Among the more important engineering projects under the direction of the firm in which Mr. Thorndike participated, may be mentioned the design and the engineering supervision of construction of the Boston Army Supply Base, one of the larger water-front terminals built during the World War for the use of the United States War Department; the Hampden County Memorial Bridge across the Connecticut River at Springfield, Mass., a monumental structure of reinforced concrete arches; and the water supply, sewage disposal, central heating plant, and other engineering features for the Town of Mariemont, in the suburbs of Cincinnati, Ohio. In all these projects and in many investigations and reports, both engineering and economic, Mr. Thorndike's contributions were valued by reason of his sound judgment and his highly analytical mind.

Thoroughness and painstaking analysis were characteristics of Mr. Thorndike's professional work and, indeed, of whatever he undertook, whether within or without the field of engineering. Inheriting, perhaps, a "legal" mind from his father, who was a lawyer of distinction, he early gave particular attention to the legal form of engineering contracts, and although without special legal training, he became an authority in this important branch of engineering work. Always keenly interested in fair play and the advancement of ethical standards, he was a most active member of the Committee of the Northeastern Section of the Society which formulated the Code of Practice, adopted in 1927 by the Society.

He was long active in the work of the Northeastern Section of the Society, of which he was President at the time of his death. For thirty-two years he had been a member of the Boston Society of Civil Engineers, serving on many of its committees and also as a Director in 1919 and 1920. He was a member of the American Water Works Association, the New England Water Works Association, and the American Society for Municipal Improvements. He frequently contributed papers to these professional societies. He was an active member of the Boston Building Congress, the aim of which is to promote improved conditions in the building industry in Boston and in New England.

Mr. Thorndike was a member of the Harvard Club of Boston, the Harvard Club of New York, and the Harvard Engineering Society. He was long a member of the St. Botolph and Engineers Clubs of Boston and of the Boston City Club, as well as of the Boston Chamber of Commerce.

The Protestant Episcopal Church and its affiliated interests, a vital part of his life, equally with his professional work, claimed much of his time, effort, and energy. From his boyhood—for fifty-two years—when in town, he occupied his place in the family pew at the historic Christ Church in Cambridge. For thirty-five years he was elected to the Vestry, and at one time was a Warden of that parish, of which he was always a most loyal supporter. He was also connected with St. Anne's Church in Lincoln, Mass., near his country home, serving on the Vestry there and giving to that parish his fostering care and intense interest.

Shortly before the war he assisted in organizing the Conference for Church Work, an annual gathering of workers in the Episcopal Church throughout the country and from abroad. Starting in a small way, the organization has outgrown the accommodations of the Harvard Divinity School, where its summer conventions were held for a time, and, in recent years, has met in the early summer at Wellesley College. For many years, Mr. Thorndike was the managing executive of the Conference; through his efforts it was incorporated; and one of the last things he did was to finish the preparation of a code of practice for the conduct of its business.

His interests extended to many enterprises for the betterment of humanity; he was a member of the Fellowship of Reconciliation and of the Church League of Industrial Democracy; he was a member of the New China Committee, and the Japan Committee; he was long active in the support of missionary work in the Protestant Episcopal Church; and he was a liberal contributor, not only to church activities, but also to many other good causes.

For many years Mr. Thorndike made his summer home at Stony Farm, in Weston (and Lincoln), Mass., and during the latter part of his life, Weston was his legal residence. He was interested in the affairs of the town, but never held office until 1926, when he was elected a member of the Water Supply Investigating Committee. In 1927 he was elected a member of the Board of Water Commissioners.

One of his chief characteristics was his love of Nature; he was never more happy than when he was at Stony Farm. He knew and loved every tree on the place and every living creature was his companion and friend. Mt. Monadnock in Southwestern New Hampshire frequently lured him to its slopes, and for many years he was a constant visitor at The Ark, at Jaffrey, at the base of this mountain. One of his pastimes, in which he was often joined by his partner, Mr. Fay, was the surveying and the mapping of the many trails on the slopes of this beautiful isolated mountain.

Mr. Thorndike remained a bachelor throughout his life. A loyal friend and delightful comrade, with qualities of sincerity, unselfishness, and earnestness rarely found, Sturgis Thorndike in his sixty years' span of life exerted a helpful influence among all with whom he came in contact, an influence far greater than he himself ever realized.

Mr. Thorndike was elected a Member of the American Society of Civil Engineers on January 6, 1915.

EDWIN HALL WARNER, M. Am. Soc. C. E.*

DIED JUNE 17, 1927.

Edwin Hall Warner was born in New York, N. Y., on February 21, 1858, the son of William H. and Anna Pamela (Conger) Warner. He attended the University of the City of New York, (now New York University), and was registered in the Polytechnic Engineering Department of the School of Applied Science.

Mr. Warner's technical career was varied and important. During the period from 1884 to 1886, he was engaged on the Mexican National and Mexican Central Railways, as Instrumentman and Assistant Engineer in charge of various construction works. During 1887 and 1888 he was Assistant Engineer and Acting Division Engineer, Maintenance of Way, for the Union Pacific Railway Company, with headquarters in Denver, Colo. In 1888 and 1890 he acted as Assistant Chief Engineer for the Seattle, Lake Shore and Eastern Railway Company (now the Northern Pacific System), with headquarters in Seattle, Wash. During this engagement he located and built 110 miles of railway from Snohomish, Wash., north to the Canadian boundary.

From 1891 to 1898 Mr. Warner was in private practice in Seattle, engaged in the development of mining properties, hydraulic power investigations, and installation of mining power plants; and for a part of the time, he acted as City Engineer for Seattle, on extensive construction of sewers. From May, 1898, to January, 1899, he was Assistant Chief Engineer for the White Pass and Yukon Railway Company in charge of location north of the Canadian Boundary. He located and built the first miles of track extending out of Skagway. Mr. C. E. Hawkins was Chief Engineer of this extremely difficult and arduous work.

From 1899 to 1903 Mr. Warner was engaged in private practice in Republic, Ore., as Engineer for the Republic Mining Company, and also on various mine investigations and installations. In 1903, he became Principal Assistant Engineer for the Columbia Improvement Company (Stone and Webster Syndicate), on the Puyallup Power development at Electron, Wash., the Engineer in charge being Mr. J. C. Cunningham who was succeeded by George S. Binckley, M. Am. Soc. C. E. This was a 20 000 h.p. installation, and during the early stages the forces under Mr. Warner's supervision included as many as 1 400 men.

In 1904 he moved to California and became Chief Engineer of the Abbot Kinney Company, at Venice, supervising the canals and bridges for this, one of the early play areas of Los Angeles. In this period Mr. Warner was associated with the late James Dix Schuyler, M. Am. Soc. C. E., and, in 1905, became Chief Engineer of the Tri-State Land Company, at Scottsbluff, Nebr. —a project for which Mr. Schuyler was Consulting Engineer. The location of 100 miles of canal, for a 1 750 sec-ft. flow, and the construction of 25 miles

* Memoir prepared by William S. Post, Assoc. M. Am. Soc. C. E., and Philip Schuyler, M. Am. Soc. C. E.

under his direct supervision, was part of Mr. Warner's accomplishment. From June, 1906, to January, 1907, he again undertook construction at Mr. Schuyler's instance. He became Assistant Chief Engineer of the Mexican Light and Power Company, for the construction of the dam at Necaxa, Mexico, where he had responsible charge of dam, tunnel, conduit, and machine shops and the construction of 22 miles of railway.

From 1907 to 1910 he was engaged in private practice in Los Angeles, Calif., designing irrigation projects, buildings, and ocean piers. From 1910 to 1916 he was Constructing Engineer for the Southern California Edison Company, with headquarters at Los Angeles. His duties included the construction of the 65 000 h. p. generating station at Long Beach, Calif., about twenty-five sub-stations, and the direct supervision of a large power tunnel on the Kern River. From 1916 to 1919 he was again in private practice, acting, among other engagements, for the American Trona Company at San Pedro and Searles Lake, Calif., in potash production.

From 1919 to 1921 Mr. Warner served as Construction Engineer for two important concrete dams in California, namely, the Kerckhoff Dam, of the San Joaquin Light and Power Company, on the San Joaquin River, and the Snow Mountain Power Company Dam on the Eel River.

In 1922, he re-entered private practice, first maintaining an office in San Francisco, and, finally, removing to Burlingame, Calif., where he and Mrs. Warner established a delightful home. It was in his office in Burlingame that he died of heart failure on June 17, 1927.

In Burlingame, Mr. Warner was also able to devote himself to civic affairs, for which his broad outlook, enjoyment of social and economic problems, fine tact, and personality, peculiarly fitted him. It is notable in the career of this engineer, absorbed in the main with the interests and perplexities of his clients, that he also gave generously to public service. An editorial, published a year before his death testifies to the thorough attention and organizing ability, which was an element of his success, and also to the appreciation of the community:

"A Word of Praise*"

"When a man has labored faithfully and well in public service, and particularly when that service has been unremunerative and has been performed in a quiet, unassuming sort of way, a word of praise is not amiss.

"Edwin H. Warner, last Monday night, tendered his resignation to the City Board of Trustees, after years of duty in the position of Secretary of the Park and Playground Commission. The plain truth of the matter is that Warner did practically all the work in connection with the functioning of the Commission and that he accomplished an amazing amount during the past two years.

"He resigned simply because the duties of the Commission had become an increasing burden, encroaching upon his private business, and because two years of free and voluntary labor at such a task certainly entitles a man to the privilege of relinquishing the responsibility to some one else.

* * * * * * * *

"Warner, on his own initiative, made practically a complete survey of all the trees in Burlingame. He consulted experts on the culture and diseases

* *Burlingame Advance-Star*, May 19, 1926.

of trees, determined the best trees for the city, and approached the entire problem in a thorough and scientific manner.

"And the main burden of the work has been the innumerable complaints and requests of property owners concerning their own tree problems. Warner maintained the consistent policy that these property owners were entitled to an immediate answer. This involved almost daily investigation on his part, the continual giving of time and energy to the work, and a large amount of correspondence. And that at no small sacrifice for a busy professional man to make.

"It is a matter of record that most men who have labored so faithfully in the service of the public come sooner or later to the disheartening realization that appreciation of their efforts is very seldom expressed by their fellow men and that the general public is relentlessly stingy with its praise. People walk miles or write page after page to kick about something, but they usually have no time or energy for commendation. Bearing this in mind, this is only an attempt to give a bit of honest praise where it is deserved."

Technically, Mr. Warner was known as a brilliant Location Engineer. In administrative work and construction his strongest forte perhaps was initiative. He possessed great organizing ability, in which he exercised a personal charm and sense of humor, which appealed not only to his friends, but to construction foremen and laborers as well. His tastes were fine; he was cultivated and very well read; and an excellent conversationalist.

One of the writers had the good fortune, as City Engineer of Ocean Park and Venice, from 1904 to 1905, to become intimately acquainted with Mr. Warner, later following him to Nebraska as an Assistant on the construction of the Tri-State Land Company Canal Project. Mr. Warner was a tall, slender, erect, handsome man, dignified and austere on first acquaintance, but possessed of a rare sense of humor and the ability to "play". He frequently organized minstrel shows, acting equally well the part of interlocutor and end-man. In short, he could play with his subordinates and at the same time retain their respect.

Mr. Warner was married on August 21, 1890, at Seattle, to Frances B. Ferguson, the daughter of Colonel David Ferguson, a distinguished figure in the Sixties in California and later of the American Colony in the City of Mexico. With sympathy and pride in her husband's engineering achievements Mrs. Warner was a constant companion in his various engagements in the Western United States and Mexico.

Mr. Warner was elected a Member of the American Society of Civil Engineers on October 4, 1893.

FRANCIS REPETTI WELLER, M. Am. Soc. C. E.*

DIED JANUARY 13, 1929.

Francis Repetti Weller was born in Washington, D. C., on February 16, 1880, the son of Michael I. and Rita (Repetti) Weller. He was educated at Mount St. Mary's College, near Emmitsburg, Md., and then attended the Columbian University, at Washington, D. C., which is now known as George Washington University. He was graduated from Columbian University with the degree of Bachelor of Science in Civil Engineering in 1899, and in June, 1900, received the degree of Civil Engineer.

From 1899 to 1903 Mr. Weller was employed in the Engineering Department of the District of Columbia, principally on bridge design, and, in addition, was an Instructor at Columbian University. In September, 1903, he entered private practice as a Consulting Engineer, with headquarters at Washington.

Mr. Weller specialized in the design of steam-electric and hydro-electric power plants, transmission lines, and water-works plants. He also always had been closely connected with public utility work. For the Municipal Service Corporation, he designed and enlarged plants in Salem, Ohio, Alexandria and Staunton, Va., and Coatesville, Pa. He was the Consulting Engineer on the design and construction of hydro-electric plants for the City of Richmond, and the City of Bedford, Va., the Watauga Power Company, at Bristol, Tenn., and the Virginia Alberene Corporation; he also made enlargements of the electric plants of the Keystone Power Corporation, at Ridgway, Pa., and the Potomac Electric Power Company, at Washington.

Mr. Weller was one of the pioneers in power development in Southwestern West Virginia and built the first power plant in that State at Logan. This plant was primarily owned by the General Utilities Operating Company, of Baltimore, and, later, by the Kentucky and West Virginia Power Company, and is now the property of the American Gas and Electric Company. Mr. Weller maintained his connections during all these changes of ownership and was responsible for many additions to the property, as well as transmission lines connecting it to plants at Sprigg, W. Va., and Hazard, Ky.

In 1917, and during the World War, he was employed by the Navy Department, Bureau of Yards and Docks, to design additions to a great many of the Navy Yards on both the Atlantic and Pacific Coasts. During this period he had a large designing force in his employ and made plans for extensions and enlargements at the Brooklyn, N. Y., Navy Yard, the Navy Yards at Philadelphia, Pa., and Norfolk, Va., the Submarine Base at New London, Conn., and a Naval Station at New Orleans, La., together with about ten other smaller projects. He was also Engineer for the United States Housing Corporation during this same period. In recognition of his excellent work for the Bureau of Yards and Docks he was made Lieutenant Commander in the United States Naval Reserve.

* Memoir prepared by M. X. Wilberding, Pres., Francis R. Weller, Inc., Washington, D. C.

In 1923 Mr. Weller was employed as Consulting Engineer for the Virginia Western Power Company and built steel tower transmission lines from Charlottesville to Staunton, Va., revamped the power house at Charlottesville, and constructed sub-stations at both Charlottesville and Staunton. The following year he built steel tower transmission lines for the same Company from Clifton Forge, Va., to Ronceverte, W. Va., and wood pole distribution lines between Goshen, Buena Vista, Lexington, and Clifton Forge, Va.

In 1925, he organized a public utility holding company known as the Interstate Utilities Corporation. This holding company had properties in Central and South Georgia and Mr. Weller was its President, as well as the President of all its subsidiary companies. These properties were afterward purchased by the Southeastern Power and Light Company, and Mr. Weller then formed the Allied Utilities Corporation, another public utility holding company, of which he was President. The principal holdings of this Company were in the State of West Virginia. The Company was dissolved in 1927 and its properties were sold to the Allied Utilities Company, of which Mr. Weller was President at the time of his death.

In the spring of 1926, Mr. Weller was employed as Consulting Engineer by the Fitkin Interests to build transmission lines in Florida and South Georgia. These consisted of steel power and wood-pole lines from Tarpon Springs, near St. Petersburg, Fla., through Inglis and Jasper to Valdosta, Ga. The installation included a great many sub-stations, and the construction work continued for a period of approximately two years.

In June, 1928, he incorporated his engineering practice under the name of Francis R. Weller, Incorporated, and, at the time of his death, he was President of this organization.

Mr. Weller was a member of the American Institute of Electrical Engineers and a Charter Member of the Washington Society of Engineers. He took a great interest in civic and charitable organizations. He was a Director of the Washington Board of Trade and President of the Roman Catholic Charities of Washington. Mr. Weller was greatly interested in the Community Chest work and just before his death was made a Vice-President of the Washington Community Chest. His active work in the Washington Board of Trade and his engineering training gave him the Chairmanship of the Water Supply Committee of that body and through it he was instrumental in bringing about the present improved conditions of the Capital City's water supply.

Mr. Weller was a member of a great many social organizations and clubs in Washington, among which were the Knights of Columbus, the University Club, the City Club, the Rotary Club, the Racquet Club, the Congressional Country Club, and the Columbia Country Club. At the time of his death he was a Director of the East Washington Savings Bank and the Federal American National Bank. He was also a director of Edmonds Art Stone Company and the Washington Wimsatt Company.

In 1911, he was married to Sallie A. Harlow, of Alexandria, Va., by whom he is survived. He also left four children, Francis Harlow, Sallie Rita,

Richard Hartman, and George Harlow Weller. Mr. Weller was always a devoted family man, and he took a great interest in his summer home which was located at Buena Vista, Pa.

He occupied the position of an outstanding citizen in the City of Washington and was greatly interested in questions pertaining to it and especially to its charity work. His passing was mourned by thousands of friends and especially by the poor and needy whom he had helped so often.

Mr. Weller was elected a Junior of the American Society of Civil Engineers on February 5, 1901; an Associate Member on November 1, 1905; and a Member on April 30, 1912.

BERTRAND THORP WHEELER, M. Am. Soc. C. E.*

DIED MARCH 20, 1928.

Bertrand Thorp Wheeler was born in Lempster, N. H., on November 25, 1863, the son of Daniel Bingham and Maria Wheeler. His boyhood days were spent in Cambridge, Mass., where he attended the public schools until he reached the age of 15. In 1879, he entered Dartmouth College at Hanover, N. H., where he took a scientific course, and was graduated in 1884 with the degree of Bachelor of Science.

Mr. Wheeler's first employment was in 1885 as Rodman with the Old Colony Railroad Company, now part of the New Haven System. In 1886, he was made First Assistant to the Chief Engineer of the "Old Colony", which position he held until 1895. During this period he had charge of second-track construction, grade separation, and other minor work. He originated and promoted the plans for separation of grades at West Fourth Street, Boston, Mass., involving a new bridge over Fort Point Channel which was among the first works of this character accomplished in Massachusetts.

From 1895 to 1896, and from 1900 to 1901, Mr. Wheeler held the office of Superintendent of Streets for the City of Boston under the Republican administrations of Mayors Curtis and Hart, making a splendid record in this onerous position. He was engaged in private practice from 1896 to 1900.

In 1902 he entered the service of the New York, New Haven, and Hartford Railroad Company as Assistant Engineer of Construction, which position he held until 1907, when he was made Engineer of Construction. During this period he had charge of the work in connection with the elimination of grade crossings at Dudley Street, and Savin Hill to Neponset (Boston), Hyde Park, Attleboro, Worcester, and other points.

From 1910 to 1912, Mr. Wheeler was also Engineer of Construction of the Boston and Maine Railroad Company. On November 1, 1912, he severed these connections and accepted the office of Chief Engineer of the Maine Central Railroad Company (succeeding the late Mr. T. L. Dunn), taking up his residence at Portland, Me. He was also Chief Engineer of the Portland Terminal Company, a subsidiary of the Maine Central. In this position he was responsible for the condition of right of way and structures on more than 1 100 miles of road, for property valued at nearly $50 000 000, and at times he had more than 3 500 men working under his direction.

During his service as Chief Engineer of the Maine Central Railroad, the line was extensively modernized and many new structures were created under his direction, some of the more notable of which were the double-tracking of the main line between Clinton and Waterville, Me., the erection of railway bridges across the Kennebec River at Augusta and Fairfield, and the construction of the Rigby Terminal Yards, in South Portland, Me.

Mr. Wheeler was a man of brilliant intellect, sound sense, and good judgment, possessing a rare faculty of analyzing a problem or looking over a plan and intuitively picking out any flaws it might contain. He was noted for

* Memoir prepared by A. H. Morrill, M. Am. Soc. C. E.

the courage of his convictions and would stand by his guns under fire. His intolerance of incompetency, the bluntness and directness of his judgments, and his ever-constant desire for results often caused him to be misjudged by those who did not know him intimately. His associates and employees could see farther than the rather brusque exterior of the man, recognizing and prizing his uniform and constant justice, impartiality, and loyalty. Among them the feeling of sadness for a friend departed is tempered by the thought that his work and his memory will long endure.

Mr. Wheeler had not been in the best of health for two years previous to his death. On February 22, 1928, he was operated on for sinus and antrum trouble at the Maine Eye and Ear Infirmary, in Portland. Although he rallied from the operation, the respite was brief and he failed gradually, passing away on March 20.

In 1888, he was married to Mabel Cole, the daughter of George H. Cole, proprietor of the American House, at Fitchburg, Mass., by whom he is survived. He also left six children, Bertrand C. Wheeler, of Chicago, Ill., Allan T. Wheeler, of Wellesley Hills, Mass., Donald B. Wheeler, of Buffalo, N. Y., Mrs. Warren B. King, Mrs. Theodore K. Thurston, and Miss Ruth Wheeler, of Portland. Mr. Wheeler was always a devoted family man and took great interest in his summer home which was beautifully located on Great Diamond Island in Casco Bay, near Portland.

He was a member of the Boston Society of Civil Engineers, the Dartmouth Club of Boston, and the Massachusetts Society of Colonial Wars and Sons of the American Revolution.

Mr. Wheeler was elected a Member of the American Society of Civil Engineers on June 30, 1910.

NISBET WINGFIELD, M. Am. Soc. C. E.*

DIED FEBRUARY 24, 1928.

Nisbet Wingfield was born at "Cloverdale", the summer home of his family, in Dade County, Georgia, on September 23, 1861, the second son of Alfred and Julia (Lea) Wingfield. He attended the University of Mississippi and also the University of Tennessee from which he was graduated in 1880.

Immediately upon his graduation he engaged in railroad engineering, continuing in that field on various railroads in the South until January, 1885, when he went to Chattanooga, Tenn., as Chief Engineer of the Water-Works. He remained in Chattanooga until 1895, when he removed to Atlanta, Ga., where he made his headquarters while engaged on the design and construction of water-works and sewerage projects at various places.

In 1898, he was called to Augusta, Ga., to take charge of the enlargement of the City Water-Works, and, later, became City Engineer and Commissioner of Public Works, holding this position until 1919, when he engaged in private practice as Consulting Engineer. While City Engineer of Augusta, Mr. Wingfield planned and constructed a system of levees for the protection of the city against the flood waters of the Savannah River. This was the outstanding achievement of his career, and will serve as a monument to him. The *Augusta Chronicle*, speaking editorially, stated:

"The security of the city against flood waters has been proven time and time again and Nisbet Wingfield was the man who suggested a levee, who planned it, supervised its construction, and its success is too well known to need accentuating".

He also planned levees for Chattanooga and for West Point, Ga.

Among the projects constructed by Mr. Wingfield are several bridges in various parts of the South, including the Archibald Butt Memorial Bridge, at Augusta, a structure of pleasing architectural design, as well as the water-works at Augusta, Montgomery, Ala., Meridian, Miss., Greenwood, S. C., and a number of smaller cities. As a Consulting Engineer he was widely known throughout the Southeast and his opinion was sought on many large projects. During the World War he was retained by the Government as Project Engineer on the construction of Camp Hancock at Augusta, and on the housing project which was commenced for the enlargement of the Navy Yard at Charleston, S. C. At the time of his death, he was Consulting Engineer for the Savannah River Electric Company, in connection with this Company's proposed hydro-electric development of the Savannah River above Augusta.

Mr. Wingfield was a man of charming personality, kind and courteous to all men; cultured and polished, a delightful companion to old and young; and a man of unimpeachable integrity. His friend, Thomas Barret, former Mayor

* Memoir prepared by James Houstoun Johnston, James J. Gaillard, and James E. Parker, Members, Am. Soc. C. E.

of Augusta, said of him that he was "a golden-hearted gentleman, an honest public official, and one of America's really great engineers; an entire city deplores his passing".

Mr. Wingfield loved his profession and was ever an honor to it. Broad-minded and liberal, he was always strict yet perfectly fair in all his dealings with those with whom he worked. The Society, in which he was always active and, in particular, the Georgia Section, in which he also took a lively interest and had served as Vice-President, have lost a valued member, and those who had the pleasure of his acquaintance and his professional association keenly regret his passing.

Mr. Wingfield was elected a Member of the American Society of Civil Engineers on May 1, 1895.

ANDREW LEWIS ACKHART, Assoc. M. Am. Soc. C. E.*

DIED MARCH 11, 1928.

Andrew Lewis Ackhart, the son of Nathan and Eva (Deyo) Ackhart, was born at Clintondale, N. Y., on April 3, 1887. He received his preliminary education in the schools of Highland and Poughkeepsie, N. Y., and, later, attended Cornell University at Ithaca, N. Y., from which he was graduated on June 22, 1911, with the degree of Civil Engineer. He supported himself at the University as a pioneer salesman of aluminum cooking utensils and by other activities.

Mr. Ackhart's first engineering engagement was as Inspector with the Farris Engineering Company, of Pittsburgh, Pa., on a concrete substructure for a 970-ft. highway bridge, which occupied his time from November, 1911, to January, 1912. Subsequently, from January to October, 1912, he held the position of Rodman and Instrumentman on extensive stadia topographical surveys with the Costa Rica and Panama Boundary Arbitration Commission.

In February, 1913, Mr. Ackhart accepted a position with Mackenzie, Mann, and Company of Toronto, Ont., Canada, in which he had charge of the design of concrete bridge foundations, retaining walls, and culverts for the Canadian Northern Ontario Railway. In May, 1913, he entered the employ of James McAlear, of Toronto, as Draftsman on heating, lighting, and ventilating installation for office buildings, hospitals, schools, and factories. From July to August, 1913, he was engaged as a Draftsman by J. H. Trommanhauser, of Toronto, on the design of machinery for a grain elevator for the Quebec Harbor Commission.

In September, 1913, Mr. Ackhart returned to Mackenzie, Mann, and Company, where he served until January, 1915, as Assistant Superintendent on the construction of round houses, machine shops, pumping stations, and all other division buildings at Fitsback, Ont., Canada. His work also included the construction of tanks, and a section house, along 100 miles of the Canadian Northern Ontario Railway.

From April to June, 1915, he had charge of the construction of a private water supply and sewerage system at Clintondale, N. Y. Having been refused for service in the Army, because of a defective eye, he "did his bit" as a Head Inspector of 4½-in. shells at various munition plants throughout Ontario, for the Canadian Inspection and Testing Laboratory, Limited. From June to December, 1916, he was engaged by the Buffalo Construction and Equipment Company, of Buffalo, N. Y., in charge of 4 miles of a catenary tower concrete foundation, and bridge abutments and culverts for the International Railway Company. During January and February, 1917, he served as Draftsman with the Castner Electrolytic Alkali Company, of Niagara Falls, on structural steel design and machinery layouts; and from February to October, 1917, as Draftsman with the Hooker Electro-Chemical Company, of Niagara Falls, on the design of a pump-house and intake for a pumping station with

* Memoir prepared by Ernst G. Kaufmann, Assoc. M. Am. Soc. C. E.

a daily capacity of 24 000 000 gal., as well as on plant extension design and a complete design for the extension of hydrogen installation.

On October 8, 1917, Mr. Ackhart accepted the position of Assistant Engineer with the Eastern Concrete Steel Company, of Buffalo, where he had responsible charge of estimates. It was during this time that he supervised the erection of the New York Central Railroad Express Station and other structures, which consisted of a grade-crossing viaduct, a concrete building, etc.

In 1919, he was engaged by The Boldt Construction Company, of Cleveland, Ohio, as Engineer and Estimator, and he held this position until his death on March 11, 1928.

On April 29, 1922, Mr. Ackhart was married to E. Louise Haner, who with two daughters, Sally Louise and Barbara Lucille, survives him.

Mr. Ackhart was a Knight Templar, a Shriner, and a Thirty-second Degree Mason, holding a great respect for this Order, of which he was a staunch supporter.

A man greatly admired and honored, he had a good word for all with whom he came in contact. He was a diligent worker and possessed a strong, magnetic personality.

Mr. Ackhart was elected a Junior of the American Society of Civil Engineers on March 4, 1913, and an Associate Member on September 10, 1918.

FREDERICK BAIRD ADAMS, Assoc. M. Am. Soc. C. E.*

DIED NOVEMBER 13, 1926.

Frederick Baird Adams was born on March 6, 1875, at Reading, Pa., the son of E. Ralph and Loretta (Loag) Adams. His education was acquired in the public schools of Reading.

In September, 1893, Mr. Adams entered the service of the Philadelphia and Reading Railway Company as Rodman, and in March, 1894, he accepted a position in the same capacity with Mr. W. H. Dechant, of Reading. From September, 1894, to June, 1895, he was a student of civil engineering at the University of Pennsylvania, following which he became Timekeeper with Mr. O. N. Wieand, on the construction of heavy masonry walls and foundations at Reading.

From January, 1896, to May, 1897, Mr. Adams served as a Clerk in the Maintenance-of-Way Department of the Philadelphia and Reading Railway Company. In 1897, he was engaged as a Transitman for the City of Reading in charge of sewer and other municipal work. Subsequently, he was appointed Engineer in charge of the construction of house sewers for the City of Reading, which position he held until April, 1900.

In July, 1900, Mr. Adams accepted a position as Assistant Supervisor for the Philadelphia and Reading Railway Company and, in December, he was appointed as Supervisor at Shamokin, Pa. In March, 1910, he was transferred to Pottsville, Pa., and on November 1, 1925, he was made Supervisor at Reading, which position he held at the time of his death.

He was a remarkably fine type of man, fond of his home and family, and active in his affiliation with the Protestant Episcopal Church. He was very popular with his business associates. He was always attentive to his duties, and showed considerable energy and capacity for work. He was particularly successful in the construction of the trackage in the Saint Clair Yards of the Reading Company, in 1910, and for several years thereafter. His general characteristics were those of a student, and he specialized at one time in bridge design.

He died on November 13, 1926, at the Homeopathic Hospital, in Reading, after a brief illness.

Mr. Adams was married on October 6, 1903, to Lulu G. Felix, of Reading, who survives him.

He was a member of the American Railway Engineering Association, the Engineers' Society of Pennsylvania, and a Certified Member of the American Association of Engineers.

Mr. Adams was elected an Associate Member of the American Society of Civil Engineers on August 31, 1925.

* Memoir prepared by John S. Goodman, M. Am. Soc. C. E.

HIRAM ELLSWORTH BALDWIN, Assoc. M. Am. Soc. C. E.*

DIED APRIL 16, 1927.

Hiram Ellsworth Baldwin was born in Youngstown, Ohio, on May 2, 1862, the son of George Simeon Baldwin and Hannah (Grey) Baldwin. Both parents were of sturdy American pioneer stock and spent their lives in Youngstown and the neighboring town of Niles, Ohio.

Mr. Baldwin's early education was obtained in the public schools of Niles. Following his steadfast boyhood desire for an engineering training, he entered the Massachusetts Institute of Technology, graduating therefrom in the Class of 1890 with the degree of Bachelor of Science.

Immediately following his graduation, he secured employment with The Brown Hoisting Machinery Company, of Cleveland, Ohio. His choice of this Company was a deliberate one, as he was greatly impressed with the developments then being carried on in machinery for the handling of ore and coal by Mr. Alexander E. Brown, the inventor. Mr. Brown had drawn around him a brilliant staff of young engineers, many of whom were to make names for themselves in later years. Mr. Baldwin was placed at once in the Engineering Office and his advance was rapid. While with this Company he not only achieved splendid success in the office, but also was sent on important engineering and erection work in Europe as well as in the United States. An especially notable accomplishment was the installation, under his direction, of the 100-ton capacity, balanced crane on the *Hercules*, of the U. S. Navy, the largest floating crane at that time.

He was appointed Chief Engineer of Special Work by The Brown Hoisting Machinery Company, and under his supervision came many developments in machinery for the rapid and economical handling of ore, coal, limestone, and other materials. Furnace hoists, bins, larries, locomotive cranes, overhead cranes, trollies, etc., were also designed and developed.

Mr. Baldwin left The Brown Hoisting and Machinery Company in 1908, and became identified as Chief Engineer with The Variety Iron Works Company, of Cleveland, Ohio. In this capacity, his work was of a most varied character, including the design of furnace hoists, open-hearth furnaces, ore and coal bins, as well as general structural steel work.

During the World War period he was instrumental in designing and building much important work for the United States Army and Navy, on which he labored indefatigably to the detriment of his health.

He resigned his position with The Variety Iron Works Company in 1922, to become Chief Engineer of The Brown Hoisting Machinery Company, with which he remained until ill health compelled him to resign. His death occurred on April 16, 1927; interment was at his boyhood home in Niles.

Mr. Baldwin was married on September 19, 1893; he had three children, one girl and two boys. One son was killed in action in the Argonne during the World War. Mr. Baldwin's widow, two children, and a sister, Lida F. Baldwin, survive him.

* Memoir prepared by E. C. Pierce, Esq., Cleveland, Ohio.

He was a member of the Cleveland Engineering Society, the National Aeronautic Association, the Cleveland Chamber of Commerce, and the Chagrin Valley Country Club.

He had a strong and forceful character, and was beloved by his friends and respected by all with whom he came in contact. His was a keen and analytical mind, and his work in his chosen profession was always well done and frequently most brilliant. His loss is a distinct one.

Mr. Baldwin was elected an Associate Member of the American Society of Civil Engineers on January 2, 1895.

HOWARD PAUL BAYLY, Assoc. M. Am. Soc. C. E.*

DIED AUGUST 22, 1927.

Howard Paul Bayly was born October 22, 1884, in Washington, D. C. He was educated in the public schools in the District of Columbia, and was graduated from McKinley Manual Training School in 1903. He received the degree of Bachelor of Science in Civil Engineering from the George Washington University in 1912. In connection with his school and college activities, it should be mentioned that Mr. Bayly took a prominent place in the field of athletics in support of his class interests. He was President of the Engineering Group in his graduating class, and was a member of the Sigma Phi Epsilon Fraternity and President of the Local Chapter. After graduation he followed the development of these same characteristics and held a prominent place in both business and social activities.

Mr. Bayly had varied experience in engineering work of a laboratory nature with Mr. Emile Berliner, in connection with the perfecting of phonograph records, in the United States and Canada. After his graduation from George Washington University he was connected with the Southern Railway Company in construction and engineering work, in charge of its transfer yards at Spencer, N. C., and other important projects in the central southern section of the country. For a time he was Assistant Engineer, with headquarters in Richmond, Va.

In 1918 he left the employ of the Southern Railway Company and joined a contracting firm, Saville and Claiborne, of Richmond. In connection with this firm's work, he was in charge of the residential development which it was carrying on, and he was also engaged in water-works and sewage disposal projects for a number of small cities in Virginia.

With the entry of the United States into the World War, Mr. Bayly was placed in charge of a part of the construction of the cantonment at Camp Lee. Following the completion of this very important project, he was in charge of the DuPont de Nemours housing and industrial project at Hopewell, Va. After the closing of the war projects, Mr. Bayly returned to the building industry in Richmond in association with the firm of Claiborne and Taylor, Contractors, enlarging his activities to include all branches of construction work.

In 1923, he left the firm and engaged in business for himself, confining his activities principally to the very best grade of residences and to more important individual projects rather than to general development work.

Mr. Bayly was married on May 7, 1914, to Alice Burritt, of Washington, D. C., who survives him.

He was a member of Dove Lodge No. 10, A. F. and A. M. The news of his sudden death on August 22, 1927, was keenly felt by his many friends and associates.

Mr. Bayly was elected an Associate Member of the American Society of Civil Engineers on April 16, 1918.

* Memoir prepared by William S. Gordon Dulin, Esq., Charleston, W. Va.

MERLIN CROSS CRAWFORD, Assoc. M. Am. Soc. C. E.*

DIED FEBRUARY 11, 1928.

Merlin Cross Crawford was born at Chilton, Tex., on August 21, 1889, the son of William B. and Alice Crawford.

He was graduated from the University of Texas at Austin, Tex., in 1912, with the degree of Bachelor of Arts, and in the following year received the degree of Electrical Engineer.

Mr. Crawford's first work after leaving the University was in the Testing Departments of the General Electric Company at Schenectady, N. Y., and Pittsfield, Mass. After having served a year with this Company he returned to his native State, where the work of reclaiming lowlands by drainage ditches and levees constructed by dragline excavators was in its infancy and where the construction of highways was creating a demand for gravel. Although he had been educated as an Electrical Engineer, Mr. Crawford foresaw the enormous amount of reclamation work in prospect and decided to cast his lot into that engineering field. He therefore joined the forces of Callahan and Crawford in the operation of gravel properties in South Texas.

His next position was with the W. E. Callahan Construction Company at Ennis, Tex., in the capacity of Superintendent in charge of a large number of reclamation projects, including parts of two of the largest projects in the State, namely, the Storage Dam near Seymour, built for the Wichita Falls Water Improvement District No. 1, and the Garza Dam, built for the City of Dallas. This connection continued until 1927, when he entered business for himself. At the time of his accidental death on February 11, 1928, which resulted from a fall from a hotel window in San Antonio, Tex., he was engaged in the beginning of his largest individual contract—the erection of the steel in the thirty-two story Smith-Young Tower.

Had it not been for his untimely death, Mr. Crawford would undoubtedly have made for himself, in an even greater degree, a record for achievement, as he possessed to a remarkable extent the qualities that make for success in the construction and engineering world. His energy was unbounded and he had the happy faculty of being able to build up an organization that was ever ready, through a spirit of unfaltering loyalty, to give to him the same whole-hearted support that he had always given to his employers. His cheerful, genial personality, and his unswerving devotion to his friends made him an always welcome addition to any group.

He was a member of Hella Temple, of Dallas, and the Dallas Athletic Club. He was also a member of Sigma Alpha Epsilon Fraternity.

Mr. Crawford was married in June, 1917, to Jean Figh, of Dallas, who survived him only one month. He left, in addition to his widow, two daughters, Jean and Patricia.

Mr. Crawford was elected an Associate Member of the American Society of Civil Engineers on May 8, 1922.

* Memoir prepared by E. S. Heyser, Assoc. M. Am. Soc. C. E.

CHARLES KENNETH CURRY, Assoc. M. Am. Soc. C. E.*

DIED SEPTEMBER 17, 1927.

Charles Kenneth Curry was born on April 15, 1895, at Jackson, Mich., the son of the Rev. and Mrs. E. R. Curry. The family lived for a time in Jackson and also in Flint, Mich., when Mr. Curry answered a call in New Jersey. From New Jersey they went to Omaha, Nebr., where Charles Kenneth Curry was graduated from High School in 1913. He entered the University of Colorado the following September, and received the degree of Bachelor of Science in Civil Engineering in 1917.

A few months later, Mr. Curry enlisted in the Army and went to France with an artillery regiment. He served in five major engagements when his superior officers recognized his ability and he was sent to the Engineer Officers School at Langres, France. From this school he received a commission as Second Lieutenant in the Engineer Officers Reserve Corps. When the Armistice was signed he was sent into Germany with the Army of Occupation. He afterward entered the University of Nancy where he studied for four months.

On his return to the United States in September, 1919, Mr. Curry was engaged as Draftsman with the American Bridge Company at Gary, Ind., for a short time until a better opportunity was presented by the Robins Conveying Belt Company, Chicago, Ill., with which he was associated until 1921. In 1921, he was employed by Allen and Garcia Company, of Chicago, and his work with this firm, both in the office and in the field, was attended with success. He spent some time in Mexico on construction work at a coal mine. In 1927, he was sent to Price, Utah, by his Company to assist in putting into operation a coal tipple which he had designed. While engaged on this work he was stricken with appendicitis and died on September 17, 1927, following an operation.

As an engineer Mr. Curry's success seemed assured. He was only thirty-two years old at the time of his death, but he had laid an unusual foundation for his work and had accomplished that which would do credit to a much older man. He was practical and possessed as well a good working knowledge of theory. He had executive ability and men liked to work with him. He was a man of high ideals and charming personality.

On August 28, 1926, Mr. Curry was married to Mary Allen, of Chicago, who survives him. He was a member of the Hyde Park Baptist Church.

Mr. Curry was elected an Associate Member of the American Society of Civil Engineers on October 1, 1926. He was also an Associate Member of Western Society of Engineers.

* Memoir prepared by T. E. Lutz, Esq., Chicago, Ill.

ROBERT WAITE DAVIS, Assoc. M. Am. Soc. C. E.*

DIED NOVEMBER 18, 1927.

Robert Waite Davis, the son of Thomas Burt and Alice Elizabeth (Finch) Davis, was born in Wartrace, Tenn., on January 16, 1890, of Welsh ancestry. He was educated in the public schools and later attended and was graduated from the Branham-Hughes Military Academy, at Spring Hill, Tenn., in 1906. He afterward worked for several years before entering the University of Tennessee, from which he was graduated in 1918.

Mr. Davis began his engineering career in 1907 as Rodman, Chainman, Mapper, etc., on land surveys. From March, 1909, to January, 1910, he was employed as Surveyor, Rodman, Concrete Inspector and Tester on lock and dam construction for the War Department, U. S. Engineer Office. From January to September, 1910, he was Assistant to the Construction Foreman, Instrumentman, and Material Chaser on plant construction for the Tennessee Coal, Iron, and Railroad Company. In 1915, he returned to that Company as Instrumentman and Temporary Chief of Party on mineral land surveys.

In December, 1915, Mr. Davis accepted a position with the Nashville, Chattanooga and St. Louis Railroad Company as Office Assistant to the Location Engineer, also acting as Draftsman, Computer, etc. He held this position until September, 1916. From June to September, 1917, he was engaged as Draftsman in the office of the Louisville and Nashville Railroad Company in Louisville, Ky.

In the fall of 1917 and spring of 1918 Mr. Davis finished his education at the University of Tennessee and immediately upon his graduation in June, 1918, he joined the U. S. Coast Guard and, later, was transferred to the Engineer Officers' Training Camp, at Camp Humphreys, Virginia, where he remained until the Armistice was signed. He was honorably discharged from the Service in December, 1918.

In April, 1919, Mr. Davis accepted a position as Resident Engineer on railroad construction with the Cuyamel Fruit Company, in Honduras, Central America. He remained in this position until October, 1919, at which time he returned to the United States. In January, 1920, he again went to Honduras and until October of that year, acted as Resident Engineer and Locating Engineer on railroad construction and location for the Cuyamel Fruit Company.

After Mr. Davis returned to the United States in October, 1920, he accepted a position as Resident Engineer on highway construction with the Louisiana Highway Commission. His first assignment was on the construction of the Federal Aid Highway between Covington and Slidell, La. This project was completed in 1921; and he was then sent to Mansfield, La., as Resident Engineer on the Evelyn-Mansfield Highway, which was completed in 1922. Each of these two projects involved grading, drainage, structures, and gravel surfacing on approximately twenty-five miles of highway. After the disastrous floods in the Mississippi Valley in the spring of 1922, Mr. Davis was sent to

* Memoir prepared by N. E. Lant, M. Am. Soc. C. E.

Poydras, La., a short distance below New Orleans, where the levees and highway had been destroyed by the Poydras Crevasse. He superintended the rebuilding of the highway at this location, the work being done on a day-labor basis and under great difficulty, due to the wet weather and heavy traffic.

Due to exposure at Poydras, Mr. Davis suffered a recurrence of tropical fever which he had contracted in Honduras. As a result of this he spent several months in the fall of 1922 at Hot Springs, Ark., trying to regain his health. In November, 1922, after recovering his strength somewhat, he returned to the Louisiana Highway Commission, at Baton Rouge, and worked in the office as Checker and Computer during the winter. In the spring of 1923, he asked to be placed on construction work again and was sent to Slidell, as Resident Engineer on the Federal Aid Highway project, extending from Rigolets, La., to Slidell and from Pearl River Station, La., to the Mississippi State line. Each of these sections was approximately six miles long. The work at Slidell required Mr. Davis to be in the swamps and marshes practically every day, both sections of this road being through a virgin country without a trail of any kind over which to drive. Considerable relocation was necessary on the northern section, from Pearl River to the Mississippi State line, and this was all carried on with dispatch and accuracy. In the spring of 1925, contracts were let for the bridges over the East Pearl and West Pearl Rivers, and as the highway construction which Mr. Davis had in charge was rapidly approaching completion, he was given charge of the construction of these two bridges, both of which were steel truss bridges on concrete piers.

Mr. Davis continued in charge of the highway and bridge construction at Slidell until August, 1925, when due to ill health he was unable to continue this work. He was given a furlough and lived at Slidell several months before moving to Birmingham, Ala., where he died of a combination of tropical diseases and inflammatory rheumatism on November 18, 1927.

Mr. Davis was a careful and accurate engineer, fully investigating all details of each problem which presented itself to him. On highway location and construction, his notes and records were always complete and reliable. His sickness and death proved a great loss to the Louisiana Highway Commission and to his fellow employees.

He was a member of the Slidell Lodge of Masons, and of the American Geographical Society. Although he was not affiliated with any church, Mr. Davis had a strong Christian character and high moral standards.

He was married in 1924, to Constance Osborne, who, with one child, a daughter, is now living in Louisville.

Mr. Davis was elected an Associate Member of the American Society of Civil Engineers on November 21, 1921.

JOSEPH FRANCIS SINNOTT DONNELLY, Assoc. M. Am. Soc. C. E.*

DIED MARCH 5, 1926.

Joseph Francis Sinnott Donnelly was born in Philadelphia, Pa., on March 25, 1884. He was the sixth son of James A. Donnelly and Marguerite A. (Pope) Donnelly, both of whom were born in Philadelphia.

After graduating from the Central High School in 1902, he entered the Towne Scientific School of the University of Pennsylvania, receiving the degrees of Bachelor of Science and Master of Science in 1906 and 1907, respectively. While at College, he was a member of the Plumb Bob, the Civil Engineering Society, and the Newman Club, and, after his graduation, he became affiliated with the General Alumni and Civil Engineering Associations, and the University of Pennsylvania Club of New York.

During the summer of 1906 Mr. Donnelly was employed as Inspector by the American Dredging Company, of Philadelphia. On the completion of his college work in 1907 he entered the employ of the American Bridge Company at its plant in Elmira, N. Y., as he had decided that structural engineering offered the most compatible field to him. In Elmira, he went through the preliminary stages of office work on structural design—tracing, detailing, etc.—remaining there until early in 1909, when he left to work on layout plans for the American Heating and Ventilator Company, of Philadelphia.

Later, in the same year, Mr. Donnelly accepted a position with the New York Central and Hudson River Railroad Company as Draftsman in the Office of the Engineer of Structures, Exterior Lines. He remained in this position about four years, detailing, designing, and checking plans of railway structures of various kinds.

In 1913, he accepted a position with the Cuba Railroad Company at Camaguey, Cuba, becoming, in turn, Chief Draftsman, Bridge and Building Engineer, and Office Engineer, which last named position he held at the time of his death.

The greater part of the thirteen years during which he was associated with the Cuba Railroad Company (which later became the Consolidated Railways of Cuba) was spent in Cuba. He also served for a time in the New York Office of the Company checking plans of structures for a new branch line and for new shop buildings. He was active in military service during the World War and was commissioned a First Lieutenant of Engineers, doing training and camp service.

During the time spent in Cuba, Mr. Donnelly was placed in charge of many important works of varied character. Extensions and branch lines were built, requiring new structures; lines were acquired, which also necessitated new, remodeled, and strengthened structures; and consolidation with the Cuba Northern Railway added greatly to the duties and responsibilities of his position.

* Memoir prepared by R. F. Patterson, Esq., Camaguey, Cuba.

Mr. Donnelly filled his several positions with the Cuba Railroad Company and elsewhere with marked ability which was attested by his steady advancement under each employer. His never-failing cheerfulness and tact, and his willingness to help other departments, as well as branches of his own, were highly appreciated by his employers and fellow workers alike.

He was a member of the American Railway Engineering Association and an organizer and Charter Member of the Knights of Columbus, of Camagüey, Cuba.

On December 27, 1919, Mr. Donnelly was married to Meta Jean Apeldorn, of Philadelphia, who, with two children, Ida Marguerite and Joseph Apeldorn, survives him.

Mr. Donnelly was elected an Associate Member of the American Society of Civil Engineers on August 31, 1915.

GUY BANKER EDWARDS, Assoc. M. Am. Soc. C. E.*

DIED NOVEMBER 4, 1927.

Guy Banker Edwards, the son of Dr. John Edwards and Myra (Banker) Edwards, was born at Gloversville, N. Y., on April 6, 1875. His boyhood years were passed in Gloversville, where he received his education in the public schools. He then attended the Clinton Liberal Institute at Fort Plain, N. Y. After his graduation from this preparatory school, he attended Philips Academy at Andover, Mass., and, later, continued his studies in Civil Engineering at Union College, Schenectady, N. Y., from which institution he was graduated in 1895.

From 1895 to 1906 Mr. Edwards held the position of Inspector of Public Works or Assistant Engineer at Gloversville. From 1906 to the spring of 1909, he was employed as Construction Engineer and Superintendent with Townsend and Fleming, Landscape Architects, of Buffalo, N. Y. During this time he was in charge of important and extensive landscape construction programs on a number of large private estates.

In 1909 he entered the employ of Mr. Warren H. Manning, Landscape Architect, in Boston, Mass. During his association with Mr. Manning he was in charge of large private estate construction work. In October of that year and during 1910, he was employed in the operation of stone quarry and stone sawing plant.

From April to December, 1911, Mr. Edwards was Resident Engineer of the Iowa Engineering Company at Clinton, Iowa, in which position he was in charge of asphaltic concrete paving and sewer system and disposal plant construction. He later accepted a most important position with F. A. Barbour, M. Am. Soc. C. E., of Boston, and E. G. Bradbury, M. Am. Soc. C. E., of Columbus, Ohio, as Resident Engineer in charge of extensive water supply developments, including a large distributing reservoir for the City of Akron, Ohio.

From July, 1915, to June, 1916, Mr. Edwards was Supervising Engineer on sewage and garbage disposal plant construction at Akron, for the late R. Winthrop Pratt, M. Am. Soc. C. E., Construction Engineer, of Cleveland, Ohio. Immediately following this engagement he accepted a position as Resident Engineer for the Cleveland and Youngstown Railroad Company in charge of important construction work, and from September, 1921, to September, 1923, he was employed by Mr. A. D. Taylor, of Cleveland, in charge of general construction work and as Resident Engineer on several housing developments in the Gogebic Range of Northern Michigan.

From September 1, 1923, to the time of his death, Mr. Edwards was in the employ of the Oglebay-Norton Company as Efficiency Engineer. The work for which he was responsible during the last two or three years was as follows: Insurance, safety, welfare and housing problems for iron ore mines under the

* Memoir prepared by Albert D. Taylor, Landscape Archt., Cleveland, Ohio.

Oglebay-Norton Company management, at the Montreal, Ottawa, Eureka-Asteroid, Berkshire, Bristol, and Feigh Mines.

During his work as a Civil Engineer, Mr. Edwards exhibited sterling qualities of mature manhood and profound knowledge of his field of activity. No one in the Engineering Profession who came in contact with him had other than the highest respect for his engineering ability, his seriousness of purpose, his principles of honesty, and the consistent application of his physical and mental energy to the completion of the tasks for which he was made responsible.

Mr. Edwards was elected an Associate Member of the American Society of Civil Engineers on March 13, 1917.

JOHN MARTYNN GRENFELL EDWARDS, Assoc. M. Am. Soc. C. E.*

DIED OCTOBER 24, 1928.

John Martynn Grenfell Edwards, the son of John and Lydia Edwards, was born in the small Cathedral City of Truro, Cornwall, England, on November 8, 1882. His father was an accountant, journalist, and artist, and was well known throughout the County of Cornwall for his work in connection with the Royal Cornwall Show. He was also a near relative of John Passmore Edwards, the well-known English journalist, newspaper proprietor, and philanthropist.

Mr. Edwards was one of seven children, he being the fifth. His father died at the age of forty-five when Mr. Edwards was ten years old. He received his elementary education privately, later attending the Truro Grammar School and the Redruth School of Mines. This was followed by a post-graduate course in Coal Mining in Wales where he won several prizes and a free scholarship for three years at the schools.

Before the completion of the mining course, the Boer War broke out, and he became a Trooper in Paget's Horse attached to the Imperial Yeomanry. He was seventeen at this time, and he received a four-bar medal, inscribed, "Cape Colony, Orange Free State, South Africa 1901—South Africa 1902".

On his return from the Boer War he found that a calamity had overtaken his family. His mother had befriended a poor woman who was dying from tuberculosis of the throat. Through the consequent infection, an only brother and three sisters died within a short period.

He resumed his studies at the mining school in Wales, and, in 1905, he accepted a position in India as Metallurgist with the Mysore Gold Mining Company. In 1908 he returned to England and worked as a Surveyor in a tin mine for the East Pool Mining Company, at Cornwall. A year later he went to the Rand, South Africa, to act as Surveyor for an underground development of the City Deep Gold Mining Company.

Mr. Edwards was engaged by the Messina Development Company, Limited, from 1910 to 1913 as Mine Surveyor, Chemist, and General Engineer in the development of a large high-grade copper mine at Messina, North Transvaal.

In 1913, he went to the West Indies as Superintendent for the Curacao Phosphate Company. He had charge of the installation of self-acting inclines and aerial ropeways, the construction of bins and wharves for shipping phosphates, the erection of buildings, machine shops, employers' houses, and native quarters, as well as the dredging of a channel from the sea to a lagoon which formed a natural harbor.

During the period of the World War, from 1915 to 1919, he accepted a commission as Lieutenant and, later, as Captain of a Tunneling Company in the Royal Engineers on the Somme. At different times on special work, he had charge of from 800 to 1 000 sappers. In the capture of Mont St. Quentin he was favorably mentioned in military dispatches by his superiors.

* Memoir prepared by L. A. Jenny, M. Am. Soc. C. E., and J. I. Leonard, Assoc. M. Am. Soc. C. E.

After the war, during 1919 and 1920, Mr. Edwards was associated with Professor Bernard W. Holman, of the Royal School of Mines, on research work in connection with the Cornish tin tailings. During this period, he did some prospecting in Spain on his own account.

The roving spirit still claiming him, he returned to Africa. For about a year he was Construction Engineer on some manganese property for the Dagwin Concession in West Africa, and for another year he was Irrigation Engineer for the Compagnie Générale des Colonies at Haut Senegal, West Africa. This Company was employed by the French Government to study the possibilities of the irrigation of the Niger Valley for the cultivation of cotton.

In 1924, Mr. Edwards went to South America, where he was employed as a Mining Engineer with the Poderosa Mining Company, in Chile, a concern engaged in the mining of copper. During the absence of the Manager for six months, Mr. Edwards was in complete charge of the mine. Remaining in South America, he later accepted an engagement with the Chile Exploration Mining Company supervising the construction of buildings and complete facilities for mining operations. Subsequently, during 1927, he worked on a hydro-electric development in Peru.

Early in 1928, Mr. Edwards went to New York, N. Y., and soon thereafter became associated with the valuation of the Interborough Rapid Transit Company. He was engaged on this work at the time of his death, on October 24, 1928, following an operation for appendicitis. He was buried with military honors in St. Michael's Cemetery, at Astoria, N. Y.

In 1919, he was married to Raymonde Helen Marie Louise Marguerite Vogues, daughter of M. Bertin Vogues, of Paris, France. M. Vogues was at one time Chargé d'Affaires in Chile and Minister of Instruction in Paris, as well as an editor of prominence and a proprietor of scientific journals in France.

Mr. Edwards was a capable engineer and held in high esteem by the members of his profession. Those with whom he came in contact were invariably impressed with his kindly and courteous bearing and his character as a gentleman. To know him was to respect him and increasingly admire his many virtues. He was a good friend, a faithful brother, and a dutiful son. He is survived by two sisters living at Carbis Bay, Cornwall, England.

Mr. Edwards was elected an Associate Member of the American Society of Civil Engineers on January 16, 1928.

FRANK ALEXANDER GIESTING, Assoc. M. Am. Soc. C. E.*

DIED APRIL 25, 1928.

Frank Alexander Giesting was born in San Francisco, Calif., on April 4, 1881. He was the son of Joseph G. Giesting and Clara Enneking Giesting. His youth was spent in the city of his birth, where he attended public and private schools until the end of his seventeenth year.

At that time, at the outbreak of the Spanish-American War, he enlisted as a private in the First California Heavy Artillery. Although perhaps the youngest member of his Company, he, even at that early age, exhibited such qualities of leadership that he arose almost immediately to the non-commissioned rank of Sergeant. He was chosen as a member of a small detachment from his Company to accompany the first Army Expedition to the Philippine Islands where he served with distinction until mustered out of the Volunteer Service in August, 1899.

In 1900 Mr. Giesting received a civil appointment in the Postal Department, at which time he returned to the Philippine Islands where he remained about two years. Resigning from the Postal Service in 1902, he returned to San Francisco and matriculated in the College of Civil Engineering of the University of California, Berkeley, Calif., at the beginning of the succeeding semester. He was graduated in May, 1906, with the degree of Bachelor of Science, and almost immediately began his professional career with the Progreso Mining Company at Triunfo, Baja California, Mexico, where he remained until July, 1907. At this time he was engaged for a short period as Transitman and, later, as Resident Engineer on construction for the Southern Pacific Railroad Company, then being built in Mexico.

In November, 1907, Mr. Giesting entered the service of the Mexican Light and Power Company, in which he acted first as Draftsman, then as Engineer and Superintendent on construction. It was during this engagement that he had complete charge of the construction of the famous Tanengo Tunnel (1 300 m. long) and the auxiliary works of this incline tunnel from Salto Chico to the power house. Coincidently, he was in charge of the 6 700-m. tunnel on the Laxaxalpan Diversion.

In July, 1912, he became General Superintendent for the engineering firm of Jacobs and Davies, Incorporated, which had taken over all the engineering work of the Mexican Light and Power Company, and served in that capacity until October, 1913. During the next two years he was engaged in private practice as Consulting Engineer in San Francisco.

From February, 1915, until April, 1917, he served as General Superintendent for the Aluminum Company of America in dam construction work at Alcoa, Tenn.

At the entrance of America into the World War, Mr. Giesting immediately resigned his position with the Aluminum Company and entered the Plattsburg Military Training Camp. His former military experiences, together with his obvious ability to lead men, resulted in his almost immediate transfer to the

* Memoir prepared by Alva F. Hughes, Min. Engr., New York, N. Y.

Training Camp for Engineers at Camp Humphrey, Va., from which he was graduated in the first class with the rank of Major of Engineers. His first assignment was at Camp Upton, Long Island, in the formation and building of which he gave prominent assistance. When the 302d Engineers of the 77th Division was formed, Major Giesting was assigned to that regiment. This regiment went overseas in March, 1918, and soon after its arrival in France, due to the promotion of his Senior Officers, he was made Lieutenant Colonel and given command of his Regiment. Under his leadership the 302d Engineers made an enviable record in the Argonne and were in the front line on Armistice Day. He had been given the rank of Colonel of Engineers and in that capacity returned to the United States with his Regiment in April, 1919, being mustered out of the service shortly thereafter.

After several months of rest, Colonel Giesting returned to the practice of his profession as General Superintendent of the Vanadium Corporation of America from 1920 to 1921, and, subsequently, with the Engineering Department of the National Carbon Company from 1922 to 1924. His next engagement was with the Delaware and Hudson Railroad Company with which he was connected from 1924 until 1926. From April, 1926, to the time of his death, he was in the service of the Port of New York Authority as Assistant to the Chief Executive Officer.

Colonel Giesting's outstanding qualities of leadership and organizing ability marked him as an exceptional member of the Engineering Fraternity. In going over his record it is interesting to recall that in his capacity as Resident Engineer for the Mexican Light and Power Company he had complete charge of the construction of what at that time was regarded as the longest incline tunnel in the world; and, further, as to his military record, that as far as is known he was the only Reserve Officer in command of a Combat Regiment of Engineers during the World War. In his character, he was quiet and unobtrusive, and always generous to a host of friends who mourn his passing. He was a great Engineer and a great Soldier.

Colonel Giesting was elected an Associate Member of the American Society of Civil Engineers on October 1, 1913.

CLARENCE MELROSE LARSON, Assoc. M. Am. Soc. C. E.*

DIED JUNE 9, 1928.

Clarence Melrose Larson was born on April 10, 1874, on a farm at Wauzeka, Wis., the son of L. C. and Mercy Celia (Dunning) Larson. When a small boy he moved with his family to Dakota and for several years, or until he was about ready for High School, attended a country school. He was graduated from the Preparatory College in Bridgewater, S. Dak., and then from the University of Washington at Seattle, Wash., in 1899, receiving the degree of Bachelor of Arts. While attending school in Seattle, Mr. Larson financed his expenses by working for the Third Street and Suburban Railway Company during the summers of 1898, 1899, and 1900. His work consisted of operating cars and assisting in the repair and maintenance of the cars and line.

In the summer of 1901 he assisted on the construction of a high-voltage transmission line for the Snoqualmie Falls Power Company and, subsequently, for ten months he was a Computer in the office of the City Engineer at Seattle, finally taking charge of office calculations for city street grading, paving, and the construction of the sewer and water system extensions.

In the summer of 1902 Mr. Larson was a Transitman on the preliminary survey for the construction of the Alaska Central Railway. In 1903, he moved to Madison, Wis., and entered the employ of the State of Wisconsin with the State Board of Assessments, as Assistant Inspector during a valuation of the property of the steam railroads of the State and a special investigation of land values and county records. At this time Mr. Larson took up engineering courses in the University of Wisconsin and, as before in Washington, financed his education by summer employment, and was graduated in June, 1905, securing the degree of Bachelor of Science in Civil Engineering. During this period he made surveys and estimates for park improvements for the Madison Park and Pleasure Drive Association.

As Assistant Resident Engineer for the National Railroad Company of Mexico, Mr. Larson was from March to August, 1905, in charge of maintaining about 500 miles of line, and constructing spurs. In March, 1906, he completed the construction work on eight miles of road, including a 446-ft. steel bridge with a 240-ft. steel arch, for the Cordoba and Huatasco Railway Company, Vera Cruz, Mexico.

Shortly thereafter Mr. Larson returned to Wisconsin and again entered the employ of the State as Assistant Engineer for the Board of Assessments and the Railroad Commission. He continued with these Departments which had charge of the valuation of railroads, street and interurban railroads, water, gas, electric, and telephone utilities, until 1909. In this year he received the degree of Civil Engineer from the University of Wisconsin.

In 1909 he again left the State employ to accept a position with the Chicago and Alton Railroad and the Toledo, St. Louis and Western Railroad Companies as Real Estate Engineer, which position he held until 1911. He then entered the employ of the New York Central Railroad Company as

* Memoir prepared by C. B. Hayden, Esq., Madison, Wis.

Engineer in field charge of the valuation of property and remained with this Company for eight months. Mr. Larson then returned to Wisconsin and again entered the employ of the State as Assistant Chief Engineer of the Railroad Commission and Tax Commission. He held this position until he was appointed Chief Engineer of the Railroad Commission in 1913 in which capacity he continued until his death.

On December 25, 1905, he was married to Mabel Claire Davenport, of Madison, Wis. He is survived by his widow, two sons, Philip and John, and a daughter, Elizabeth.

In the discharge of his regular duties with the Railroad Commission, Mr. Larson had complete charge of the valuations of the utilities and railroads of the State where rate matters were involved and also when security issues were sought. He also had under his general supervision the formulation and enforcement of the service and safety orders of the Commission.

From time to time he was a member of important organizations and committees having in charge matters of great public concern; he was, at the time of his death, a Member of the Transportation Survey Committee of Milwaukee, Wis., and of the Board of Directors of the Provident Building and Loan Association. He had also retired from a local Traffic Committee shortly before his last illness. He took an active part in all matters relating to the Engineering Profession and was a Past-President of the Technical Club of Madison.

Mr. Larson had a modest, unobtrusive nature. Loved by all who knew him, he was ever cognizant of merit in others, but always reluctant to assume or accept honor or preferment. His capacity for work was enormous, and he was always fully equal to any task which he might accept. Keen of intellect and unusually observant of detail, he seldom failed to cover completely and sift thoroughly the most involved and trying problems, and when it was necessary to recall the details of a question which might not have been considered for a long time, perhaps not in years, he could depend on his memory to produce the facts. His ability was recognized at the University of Washington when he was elected to Phi Beta Kappa.

Mr. Larson delighted in music, and was for years a member of the Mozart Men's Chorus; he was quite a linguist, having a general knowledge of Spanish, French, Norwegian, and German. He was a Thirty-second Degree Mason and a member of the Shrine. He was also a member of the Phi Gamma Delta Fraternity.

Mr. Larson was elected an Associate Member of the American Society of Civil Engineers on October 7, 1908.

WALTER HARLAN LECKLITER, Assoc. M. Am. Soc. C. E.*

DIED JANUARY 29, 1928.

Walter Harlan Leckliter, the son of John H. and Cornelia (Hanks) Leckliter, was born at Carbon, near Corning, Iowa, on November 19, 1883. He was graduated from the Corning High School in 1904, and from Iowa State College, at Ames, Iowa, in 1909, with the degree of Bachelor of Civil Engineering. In the interval between his Freshman and Sophomore years in college he was engaged with a surveying party on construction work for the Missouri Pacific Railroad Company, in Southern Missouri. He stood well in his studies at college and in his Senior year was elected to the Tau Beta Pi Fraternity.

After his graduation, in July, 1909, Mr. Leckliter entered the employ of the Chicago, Burlington, and Quincy Railroad Company, on maintenance work in Wyoming. He began this engagement as Rodman and was subsequently advanced to the position of Head of Party.

Previous to this time, he had taken and passed a civil service examination, and in May, 1910, he received a Government appointment to the Philippine Islands, where for three years he built roads at Baguio, the Summer Capital.

His work in the Philippines also included a several months' engagement on preliminary surveys for irrigation, highways, and bridge construction. In connection with the latter, he designed bridges and culverts of reinforced concrete and harbor improvements which consisted of docks, sea walls, and other similar structures, for the City of Samboanga. He was also in charge of the office of the Bureau of Public Works in Baguio, which office regulated and controlled all buildings, grounds, roads, water-works, telephone, and sewerage systems, etc., for the city. He also had charge of an extension of the water-works system, of which the estimated value was $20 000.

In 1913 Mr. Leckliter returned to the United States, and was employed as the first County Engineer of Adams County, Iowa. In this capacity he was engaged in road and bridge construction. In 1914, he entered private practice and began contracting for paving, with headquarters in Des Moines, Iowa. Later, he was employed as Superintendent of Construction with Akin and Flutter, General Contractors, until 1920, when he again engaged in private practice as a contractor, principally in grading and paving in Southern Iowa and Missouri. He continued in this work until ill health necessitated his retirement in 1927. He died on January 29, 1928, at his home in Des Moines.

In his school days he was quite an athlete; he was Captain of his High School football team, although he never "made" the 'Varsity eleven. One of his college papers called him "a leader on the Campus". Besides Tau Beta Pi his only social memberships included a college Fraternity, "Adelante", and the local militia company at Corning.

Mr. Leckliter's character is perhaps best indicated by what other people thought of him. As a supporter of a widowed mother, and of a younger sister, whom he sent through college, he was conscientious to the largest degree.

* Memoir compiled from information on file at the Headquarters of the Society.

Even during his last illness, he was not immune from the demands of his wide friendship. One of his former laboring men wrote that he was out of work and tramping through Louisiana, and closed with "I know that if you only knew how much I needed money, you would send me some". At that time, he was besieged with financial demands because of the many expenses incidental to a long illness; yet he told his wife to mail the man a check.

One of his business partners has said of him, "There was a square man. I have traveled over all this country with him and there was nothing crooked in him." To his many friends, to his family, to whom he meant so much, his loss is great. Such a man must indeed be counted a success in life.

He was married on March 2, 1918, to Helen Widner, of Corning, Iowa. Besides his widow, he is survived by the following members of his family: His mother, Mrs. C. J. Leckliter, two brothers, Mr. Ralph E. Leckliter, of Corning, and Mr. Oliver L. Leckliter, of Des Moines; and four sisters, Mrs. Alma Bilderback, of Council Bluffs, Iowa, Mrs. Katherine Farrar, of Richmond, Calif., Mrs. Charles McGhee, of Steins, N. Mex., and Mrs. Arthur W. Miller, of LaGrange, Ill.

Mr. Leckliter was elected a Junior of the American Society of Civil Engineers on October 1, 1912, and an Associate Member on August 31, 1915.

ALFRED LEVY, Assoc. M. Am. Soc. C. E.*

DIED MARCH 23, 1928.

Alfred Levy was born at Darlington, England, on April 26, 1878. He received his education in England, after which he served an apprenticeship from September, 1892, to April, 1899, with Mr. F. W. Lyall, Civil Engineer, of Darlington, in the surveying and construction of railways, water-works, roads, etc. His services were retained by that firm until April, 1900, when he was engaged as Assistant Engineer under Sir James Bell on the Lancashire, Derbyshire, and East Coast Railway, on maintenance and new works until April, 1903. In May of the same year he was engaged by the Consulting Engineers for the Cape Government Railways (Messrs. Gregory, Eyles and Warring) and sent to South Africa under contract as Assistant Engineer on the survey and construction of the Indwe-Maclear Line (105 miles in length); he was in charge of 30 miles of this road from earthwork to plate-laying. In May, 1906, he went to England on furlough after which he again returned to South Africa.

From February to August, 1907, Mr. Levy was Assistant Engineer on the Cape Government Railways under Mr. M. D. Robinson, and was employed on the Coastal Section of the Mossel Bay-George Line (33½ miles in length). He had charge of all outside work necessary to complete construction, including setting out, testing of foundations, and the erection of a steel-pile bridge, 650 ft. long. He subsequently returned to England where he was again employed by Mr. E. W. Lyall, of Darlington. Among other engagements he carried out a survey for reclamation work in the Tees Estuary for the Cargo Fleet Ironworks.

In July, 1908, Mr. Levy was appointed Chief Engineer of the Ferrocarril Central Dominicano, Puerto Plata, Dominican Republic (under Mr. H. Gibson). He acted for six months as General Manager, returning to England in 1912.

In January, 1913, he was appointed to the Staff of A. M. Tippett, Engineer-in-Chief of the South African Railways. He was employed for four months on grade and curve improvements on the South-Eastern Line and was then transferred to the Benoni-Welgedacht Line.

When the World War broke out in 1914, Mr. Levy was employed on the survey and construction of the strategical line from Prieska to Upington, in connection with the German Southwest Campaign. During 1915 he was engaged for eight months on reparation work, after serious washouts, on the Bandolier Kop-Messina Line.

In 1916 he was transferred to Ladysmith, Natal, to take charge of the remodeling of the Passenger, Freight, and Locomotive Yard. Heavy earthwork was involved as well as a double-track steel bridge over the Klip River. In 1918 he went to Johannesburg as Assistant Superintendent of Maintenance. The following year he was transferred to Bloemfontein and East London and

* Memoir prepared by Jervis Gibbon, Dist. Engr., South African Rys. and Harbors, Johannesburg, South Africa.

again to Johannesburg in 1922 in the same capacity. At the time of his death, Mr. Levy had been appointed System Engineer, with headquarters at East London, but had not actually taken up the work.

He always insisted on a high standard of track maintenance and was responsible for numerous improvements for the purpose of eliminating sharp curves and recentering, to allow for transition curves on the sections on which the fast trains ran. His reports on engineering work were concise and his recommendations particularly sound.

He had made a special study of the problem of soil erosion and was most successful in the methods he used for its prevention. His estimates for new works and programs for re-laying, etc., of permanent way were most carefully prepared as to quantities and prices, and local conditions were always allowed for in the price of labor. He had the faculty of preparing plans in such a way as to convey the information required without elaborate or superfluous detail.

In making Staff changes, Mr. Levy was most considerate as to educational facilities for the children of the men concerned. Fortunately for the Engineering Profession, he has handed on these splendid traditions to the younger engineers who have had the privilege of working with him.

On December 31, 1912, Mr. Levy was married to Florence Thomas, of Northallerton, Yorkshire, England, who survives him, together with two children, a daughter aged 14 and a son aged 11. He was essentially a family man, and was devoted to his wife and children.

"So many worlds, so much to do,
So little done, such things to be,
How know I what had need of thee,
For thou were strong as thou wert true!"

Mr. Levy was elected an Associate Member of the American Society of Civil Engineers on December 5, 1911. He was also a Member of the South African Society of Civil Engineers.

VIRGIL SAMMS ONSTOTT, Assoc. M. Am. Soc. C. E.*

DIED NOVEMBER 11, 1927.

Virgil Samms Onstott, the son of Dr. Elmer Onstott and Emma J. Onstott, was born on September 22, 1897, at Saltsburg, Pa. He was graduated from Saltsburg High School in June, 1915, and in the fall of that year he entered Kiskiminetas Springs School from which he was graduated with high honors in 1917, winning a medal for proficiency in mathematics as well as a scholarship conferred by the Cornell Alumni Association of Western Pennsylvania.

In September, 1917, Mr. Onstott entered Cornell University at Ithaca, N. Y. He enlisted in the Students' Army Training Corps, on October 8, 1918, and was assigned to Company H of the Reserve Officers Training Corps, from which he received his discharge the following year. In 1921, he was graduated from Cornell University with honors and a degree in Civil Engineering.

Subsequent to his graduation he was employed by the Pennsylvania Department of Highways first as Inspector and, later, as Senior Inspector in charge of various kinds of pavement construction work. In 1924 he was engaged as Assistant Resident Engineer in charge of State Highway construction in Beaver County, Pennsylvania, with headquarters at Pittsburgh and at Rochester, Pa., which position he held until his death.

Mr. Onstott was a member of the Presbyterian Church of Saltsburg with which he was affiliated from his early youth. On December 26, 1923, he was commissioned as Second Lieutenant in the United States Engineer Officers Reserve Corps.

He is survived by his widow, Margaret Anne Onstott, and an infant son, also his parents and one brother, Howard K. Onstott, of Cleveland, Ohio, and one sister, Mrs. G. G. Waite, of Toronto, Ont., Canada.

Mr. Onstott not only had all the qualifications of a good engineer, but a character which presaged his success. His loyalty and his steady purpose of co-operating in any undertaking which was assigned to him, commanded the respect and admiration of his associates. His calm judgment and unpretentious manner were noticeable in everything he undertook so willingly. It was, therefore, only natural for him to analyze his problems thoroughly—a characteristic so essential in a good engineer. His passing is a distinct loss to the profession.

Mr. Onstott was elected an Associate Member of the American Society of Civil Engineers on March 14, 1927.

* Memoir prepared by J. L. Herber, M. Am. Soc. C. E.

JAMES MADISON PORTER, 3d, Assoc. M. Am. Soc. C. E.*

DIED JUNE 3, 1928.

James Madison Porter, 3d, the son of James Madison Porter, Jr., and Ruth Pierson (Cook) Porter, was born in Easton, Pa., on May 10, 1864.

He was descended from a long line of distinguished ancestors and his family has been prominent in the making of Pennsylvania history. His father, James Madison Porter, Jr., an eminent lawyer and distinguished diplomat at the Courts of Vienna and Berlin, was a Thirty-third Degree Free Mason and Grand Master of the Pennsylvania Grand Masonic Lodge. His grandfather, James Madison Porter, was for many years one of the leaders of the Pennsylvania Bar and was President Judge of the 23d Judicial Circuit which comprised at that time the Counties of Wayne, Pike, Monroe, and Carbon. Judge Porter was appointed Secretary of War by President John Tyler, but the appointment was not concurred in by the United States Senate. He was one of the founders and the first President of the Board of Trustees of Lafayette College, at Easton, a position which he held for twenty-five years, serving as Lecturer on Jurisprudence and Political Economy without compensation during the entire period.

Judge Porter's father, the great-grandfather of James Madison Porter, 3d, was Maj.-Gen. Andrew Porter, Commissary General of the American Forces during the Revolutionary War. He was offered the post of Secretary of War in the Cabinet of President James Monroe, but owing to the infirmities of age, declined to serve. General Porter had six sons, all of whom became prominent in National life: David Rittenhouse was Governor of Pennsylvania; George Bryan was Governor of Michigan; William Augustus Porter, LL.D., the son of David Rittenhouse, was District Attorney for the City of Philadelphia, Pa., and afterward Judge of the Court of the Alabama Claims; and another son of David Rittenhouse was Maj.-Gen. Horace Porter of Civil War fame. Rear-Admiral David Porter, U. S. N., Gen. Fitz-John Porter, and Gen. Andrew Porter were all collateral relatives of James Madison Porter, 3d.

Mr. Porter attended the public schools of Easton until 1879, when, following the death of his father, he, with his mother and sister, moved to Hackettstown, N. J. He prepared for college at the Centenary Collegiate Institute of Hackettstown and Blair Academy at Blairstown, N. J., and entered Lafayette College, in the autumn of 1882. When he went to Easton, he lived with his grandfather in the house that continued to be his residence until the time of his death.

In June, 1886, he was graduated from Lafayette College with the degree of Civil Engineer. Following short engagements with Davis and Grier and the Lehigh Valley Railroad Company, he entered the service of Tippett and Wood, Phillipsburg, N. J., as Chief Engineer. His duties in this position covered the design and construction of tanks and stand-pipes. He retained

* Memoir prepared by Lynn Perry, M. Am. Soc. C. E.

this connection for the remainder of his active career, even after he took up his college duties. During this time he was engaged on the design and construction of many steel structures. One of the most important of these was the cantilever bridge connecting Easton and Phillipsburg, referred to in many articles and books on bridge engineering. This bridge which will remain as a monument to his genius for many years, was built in 1895, and has no top chord bracing. Mr. Porter was well-known as a consulting structural engineer and, during his active career, he designed many highway and trolley bridges as well as many stand-pipes and elevated tanks. He was a pioneer in the development of pressure filters.

In 1890, on the death of Professor Joseph G. Fox, Head of the Civil Engineering Department at Lafayette College, Mr. Porter was appointed Instructor in that Department and became Adjunct Professor of Civil Engineering on June 25, of the same year. On October 22, 1891, he was made Professor of Civil and Topographical Engineering, in charge of the Department, and in 1908, was appointed Professor of, and Director of the Division of, Civil Engineering. He occupied this chair until 1916 when he resigned after completing twenty-six years of service to his Alma Mater. Many generations of Lafayette men remember his outstanding qualities of leadership, teaching ability, sterling character, and genial disposition.

Professor Porter was active in church as well as technical, business, and civic affairs, and was a leader in every undertaking with which he was connected. He was a trustworthy friend of his colleagues and respected and beloved by all who were privileged to know him and who came into contact with him.

He served as Vice-President and President of the General Crushed Stone Company; Secretary of the American Society for Testing Materials; Director of the Easton Trust Company; President and Director of the Easton Tau Chapter of Zeta Psi; Vice-President and Director of the Northampton Country Club and the Pomfret Club. He was a member of the University Clubs of New York, N. Y., and Philadelphia, Pa., and the Automobile Club of America.

He was married on November 15, 1888, to Mary Virginia Drake, the daughter of Samuel Arndt Drake, of Easton, who, with an only son, James Madison Porter, IV, survives him.

Professor Porter was elected an Associate Member of the American Society of Civil Engineers on May 4, 1892.

JOHN GANSOVERTTE ROSE, Assoc. M. Am. Soc. C. E.*

DIED OCTOBER 27, 1927.

John Gansovertte Rose was born on April 11, 1878, in Hutchinson, Kans. He was graduated from the Hutchinson High School, and, later, from Nickerson College, where he completed a post-graduate course in 1903, and subsequently taught for three years. After holding various engineering positions until 1907, he entered the University of Colorado at Denver, Colo., from which he received the degree of Bachelor of Science in Civil Engineering in 1911.

During his vacation periods in 1909 and 1910, Mr. Rose assisted in both field and office for the firm of Field, Fellows, and Hinderlider, of Denver, Engineers for the Orchard Construction Company. This project includes 12 000 acres of land near Grand Junction, Colo. The irrigation system embraces a movable diversion dam 380 ft. long, extending across the Grand River; 9 miles of power canal, about 6 miles of which were bench and trestle flume; a hydraulic power plant consisting of four units of Leffel turbines, directly connected with centrifugal pumps, two of which operated a lift of 75 ft. and the other two, a lift of 125 ft.; 40 miles of distributing canals; 1 800 ft. of concrete-lined tunnel; and various other structures, such as inverted siphons, concrete culverts, highway bridges, waste-gates, etc. The estimated cost of this varied construction work was $1 250 000.

In June and July, 1911, Mr. Rose served as Engineer for the Badita Reservoir Company for which he made stream measurements and hydraulic investigations. He also submitted reports on a reservoir site in Southern Colorado.

From September, 1911, to May, 1912, he was employed by the Goldsborough Company, Engineers for the Laramie Water Company of Laramie, Wyo., on a 150 000-acre project. During this period he acted as Instrumentman on the preliminary and final location of canals having capacities of 1 000 sec-ft. and less. In June, 1912, he became Assistant Engineer in charge of the construction of canals for the Goldsborough Company. One of these, a 350 sec-ft. canal, 9 miles long, was located in a rough mountainous country covered with a dense growth of pine timber, the final estimated cost of construction of which was $168 000.

In November, when work on the canal had to be abandoned on account of the weather, Mr. Rose entered the employ of the Kansas City Structural Steel Company, of Kansas City, Mo. During the remainder of this year, and until May, 1913, he acted as Structural Detailer on smelter and railroad buildings, including stairways and hip-and-valley work.

From June, 1913, to October, 1915, he served as Engineer and Draftsman with the Consolidation Coal Company of Jenkins, Ky., making title maps of 100 000 acres of coal lands in the Cumberland Mountains. He also supervised the compilation of a complete index system of all notebooks (approximately 2 000), maps, and records of these lands, which had been collected throughout a period of forty years.

* Memoir prepared by W. L. Prouty, Assoc. M. Am. Soc. C. E., and A. E. Palen, M. Am. Soc. C. E.

From November, 1915, to March, 1916, Mr. Rose served as Draftsman for the Kennicott Company of Chicago, Ill. This Company was engaged in the manufacture of water softeners, rapid filters, boilers, gas retorts, etc., which involved considerable pipe-fitting work, straight and spiral stairways, and varied similar construction, on all of which he had assignments.

In March, 1916, he became Draftsman for the American Bridge Company at Gary, Ind. During his engagement with this firm, his work included the preparation of shop drawings for highway bridges and for a large blast furnace and rolling mill.

In September, 1916, he was appointed Assistant Engineer with the Atchison, Topeka, and Santa Fé Railway Company, continuing until June, 1918, on the location survey of a double-track line, 65 miles long, between Carrollton and Moberly, Mo., to join the surveys of the Burlington Road for a new joint railway between Kansas City and St. Louis, Mo. This location was controlled by a definite list of limitations, such as (1) distance evaluated at $200 000 per mile; (2) curvature evaluated at $480 per degree; (3) rise and fall evaluated at $700 per ft.; (4) maximum grade, eastbound track, 0.3%; and (5) maximum grade, westbound track, 0.4 per cent.

As may be inferred, this project involved some heavy construction work (single cuts amounting to 500 000 cu. yd.), such as an under-crossing of the Wabash Railroad Terminal Yards at Moberly, and also included a large reservoir for water supply. Messrs. V. C. Coffey, Location Engineer, and Woodbury Howe, Chief Location Engineer, were in charge of this unusually complicated undertaking.

During June and July, 1917, Mr. Rose located a line, about 70 miles long, on the plains of Western Kansas, between Satanta and the Colorado State Line. From August to November, he made a close preliminary survey of a line between Kingfisher, Okla., and Shattuck on the Texas border (a distance of about 135 miles). From November until the early spring of 1918, he located a line (14 miles long) between Cassoday and Bazarr, in Eastern Kansas, and a belt line, with two alternate locations, around the Town of El Dorado, Kans., in order to relieve the traffic from the adjoining oil fields. Messrs. J. C. Beye acted as Location Engineer and H. C. Steward, as Chief Location Engineer, on the Kansas and Oklahoma surveys.

This continued engagement with the Santa Fé Railway Company gave Mr. Rose a broad and valued range of experience in railroad location. His duties during the two years of service covered all phases of the work required in making surveys, maps, profiles, estimates, and reports.

From February to June, 1918, he was associated with the United States Reclamation Service on the Rio Grande Project, at El Paso, Tex., in responsible charge of canals, farm laterals, and topographic surveys. He also made estimates of the cost of reconstruction and co-ordination of 40 miles of some old Spanish canals in the El Paso Valley.

During the summer he served as Assistant Engineer with the Idaho Irrigation Company of Shoshone, Idaho, in charge of a system of distribution canals and the replacement and installation of concrete head-gates, checks, drops, etc.

During September and November, he was engaged with Lyman E. Bishop, M. Am. Soc. C. E., of Denver, Colo., as Resident Engineer in charge of construction of canal structures for the Meadow Farms Company, at Hardin, Colo.

In March, 1919, Mr. Rose was engaged as Chief of a Road Survey Party with the United States Bureau of Public Roads, District No. 3, with headquarters at Denver, in charge of the Deadwood-Hotsprings Project in the Black Hills in Lawrence, Pennington, and Custer Counties, South Dakota. This project was about 70 miles in length. Location surveys were made over about 60 miles, and construction was then in progress on approximately 16 miles of the road. His duties embraced the location and design of the road and all its structures, as well as supervision of construction, part of which was done under contract and part by force account.

In addition to this work, Mr. Rose was called on to consult and advise with local and county officials, as well as State and Forest Service officials, in various road matters relative to the Federal Aid Road Act. In this connection he had an opportunity to inspect numerous types of road construction and materials, to advise in matters of administrative organization and methods of management, and to inspect and check plans for other projects. As a matter of special interest it was his privilege to conceive and develop a new method of mass diagram which also serves as a haul sheet and furnishes an excellent means of balancing cut and fill quantities. This graph has been used extensively throughout District No. 3, and has received much favorable comment.

In 1921, Mr. Rose was appointed Assistant Highway Engineer and was sent to Washington, D. C., to receive special training as Materials Engineer on Federal Aid construction in the States of Wyoming, Colorado, and New Mexico. He held this position until his death.

Mr. Rose was married on September 15, 1912, in Denver, to Marie K. Venemann who, with four children, Jennie May, Helen Marie, Charles R., and Richard W., survives him. He is also survived by two sisters and two brothers.

Mr. Rose was a member of Union Lodge No. 7, A. F. and A. M., of Denver. He was also a member of the Colorado Section of the Society, and his death was felt as a personal loss to every member of the Section and to all who were in any way brought into contact with him.

A man who was loved by all who knew him, he was identified by his kindly ways as a genuine friend to his many acquaintances and business associates. In his home, he was a true husband and a real father; in the business world, he was a capable and efficient engineer.

Mr. Rose was elected an Associate Member of the American Society of Civil Engineers on April 14, 1919.

MICHAEL HEALEY RYAN, Assoc. M. Am. Soc. C. E.*

DIED SEPTEMBER 29, 1927.

Michael Healey Ryan was born on March 30, 1876, at Beverly, Mass., the son of Michael C. Ryan and Annie (Healey) Ryan. He pepared for college at the Lynn, Mass., Classical High School. In 1895, he entered the Lawrence Scientific School of Harvard University where he completed the four-year course and received the degree of Bachelor of Science in Civil Engineering with the Class of 1899.

During the summer of 1899 and from January to August, 1900, Mr. Ryan was Assistant Instructor in Surveying in the Lawrence Scientific School. He commenced and finished his active professional career on the New York Subway work. This service began on the construction of the first subway in New York City, on September 20, 1900, as Assistant Engineer on the Rapid Transit Engineering Staff which was organized by William Barclay Parsons, Hon. M. Am. Soc. C. E., Chief Engineer of the Board of Rapid Transit Railroad Commissioners, and was continuous from this time for nearly 19 years, until his resignation on March 31, 1919.

During the first seven months he worked in the General Offices of the Engineering Department. He started his field work in May, 1901, on the Lenox Avenue Subway (Section 9, Contract 1) in charge of a line and grade party. About two years later (March, 1903), he was given charge, under the Section Engineer, of the field parties, the inspection of construction, and office work on the Lenox Avenue Subway between 135th and 145th Streets, and the Manhattan side of the Harlem River Crossing (Section 9-A, Contract 1). This subway work was varied in character. It included steel beam, reinforced concrete, arch, tube, and concrete retaining wall construction, and also the Manhattan Approach to the Harlem River Tunnel crossing. On the completion of this work Mr. Ryan was put in charge of surveys for some routes of proposed new subways, on which duties he was engaged until the Board of Rapid Transit Railroad Commissioners was succeeded by the Public Service Commission on July 1, 1907.

Under the new Commission Mr. Ryan made a number of investigations and reports with respect to the service conditions on the surface-car lines in New York City. Later, he was employed in arbitration matters in connection with the settlement of the claims on Contract 1 for the completion of the first subway.

When construction on the Dual Subway System was begun, he was assigned to the Seventh Division as Assistant Division Engineer. He was in direct charge, under the Division Engineer and Chief Engineer, of the work of the first Sub-Division of the Seventh Division from its beginning to its completion; and, later, he was also in charge of finishing the work of the second Sub-Division. These two sub-divisions included nearly 6 miles of new subways, the construction of which cost approximately $20 000 000.

* Memoir prepared by D. L. Turner, M. Am. Soc. C. E.

The first Sub-Division work covered the lower portion of the Interborough Seventh Avenue Subway in Manhattan from the Battery through Greenwich Street, Varick Street, West Broadway, and Seventh Avenue to 17th Street (Sections 1-A, 1, 2, 3 and 4 of Routes 4 and 38). It included the construction, under difficult conditions, of a connection with the existing subway in Battery Park. In Battery Park, along Greenwich Street and West Broadway, the subway ran under the Manhattan Elevated structure. It was a two-track subway to Chambers Street and a four-track line from there on.

The second Sub-Division work was started and almost finished under another Assistant Division Engineer, but Mr. Ryan completed it. It included the building of the two-track Interborough Subway in Manhattan from a connection with the Seventh Avenue Line at West Broadway, thence under Park Place, under the Post Office Building, across Park Row, under the original subway and under Beekman and William Streets (Sections 1 and 2 of Route 48). This was one of the most difficult sections to construct on the entire Dual System. In Brooklyn, the second Sub-Division included the new subway from a connection with the Fourth Avenue Subway, at Fulton Street, thence along St. Felix Street and Flatbush Avenue to Malbone Street (Sections 1, 1-A, 1-B, 2-A of Route 12). This subway was constructed for operation by both the Interborough Company and the New York Rapid Transit Corporation. Where it is jointly operated, it is a six-track subway. The remainder of the line consists of two tracks. The two-track line is known as the Brighton Beach Connection and is operated by the New York Rapid Transit Corporation. The four-track line is a part of the Eastern Parkway Line operated by the Interborough Rapid Transit Company. One section of the line is constructed under the Atlantic Avenue Terminal of the Long Island Railroad.

During his entire contact with this great work Mr. Ryan was strong and active both in body and mind. He had a high sense of responsibility and devotion to duty. His work was always eminently satisfactory, and he was held in uniformly high regard by all his associates.

On the completion of the subway work under his charge, Mr. Ryan resigned from the service of the Commission to complete his study of law. From 1909 to 1912 he was registered in the New York University Law School. During 1919 and 1920 he was a student at the Harvard Law School and, in 1922, he was graduated with honors from the Boston University Law School. He was admitted to the Massachusetts Bar, but never practiced.

In the later years of his life Mr. Ryan devoted a great deal of time to travel. He died suddenly in Boston, Mass., from a heart attack, on September 29, 1927, a few days before he was to sail for Europe on another extended trip abroad. He never married.

Mr. Ryan was elected a Junior of the American Society of Civil Engineers on December 3, 1901, and an Associate Member on April 1, 1908.

GEORGE CHRISTIAN SCHOENBERGER, Assoc. M. Am. Soc. C. E.*

DIED SEPTEMBER 24, 1928.

George Christian Schoenberger was born in Buras, Plaquemines Parish, La., on January 17, 1875, the son of George M. and Mary (Walker) Schoenberger. He received his early education at Buras, and subsequently entered the Louisiana State University at Baton Rouge, La., from which he was graduated in 1898, receiving a Bachelor of Science degree in Civil and Mechanical Engineering. During his college career, he was a member of the famous football team of 1896, took an active interest in the military life of the college, serving as Major during his Senior year, and was a member of the Sigma Alpha Epsilon Fraternity.

Shortly after his graduation, Mr. Schoenberger entered the employ of the State Board of Engineers of Louisiana, which, in later years, was to have him as its Chief Engineer during a most trying period.

He began his professional career as Rodman, and later, was promoted to be Levee Inspector on the Red River. After a year with the State Board, he resigned to accept a position with the Houston Irrigation Company as Assistant Engineer in charge of irrigation work in Southwestern Louisiana. This engagement was followed by short services as Rodman and Inspector with the United States Engineer Office, and as Field Superintendent with the Mississippi Construction Company, engaged in levee work.

In November, 1901, Mr. Schoenberger returned to the State Board of Engineers as Surveyor in charge of party on location and construction of levees. Just a year later, in November, 1902, he was appointed Junior Engineer with the U. S. Engineer Office, New Orleans, La., in charge of levee construction in the Lafourche and Pontchartrain Levee Districts. The following six years were spent in this capacity. During a part of this time, Mr. Schoenberger and his family made their home in Natchez, Miss., where they enjoyed the society of a host of friends.

Although Mr. Schoenberger rather frequently changed positions during this early period, his work always was related to river control, in some of its varied phases. He must have realized early that this was to be his life work, and that a broad training embodying all its branches was highly desirable. Consequently, in October, 1908, he again changed positions, this time going to the Mississippi River Commission, 4th District, District Engineer's Office, at New Orleans, with which he stayed until 1917, when he was appointed a Member of the Board of State Engineers.

During the high-water periods of 1912, 1913, 1915, and 1922, much of the responsibility for keeping intact the levee lines in the 5th Louisiana and Tensas Basin Levee Districts was Mr. Schoenberger's, and due largely to his management in rescue work after the break at Fagette, La., no lives were lost. It was during this flood that Mr. Schoenberger first began to attract attention by his consummate skill and untiring efforts.

* Memoir prepared by Gervais Lombard, Esq., New Orleans, La., and A. F. Jacobi, Assoc. M. Am. Soc. C. E.

From the time in 1917 that he was appointed a Member of the State Board of Engineers by Governor Ruffin G. Pleasant, until his death, Mr. Schoenberger was constantly with the State Board, having been re-appointed in 1920 by Governor John M. Parker, and in January, 1925, being named Chief Engineer by Governor Henry L. Fuqua to fill the vacancy caused by the death of the late Frank M. Kerr, M. Am. Soc. C. E.

The record flood of the Mississippi River and its tributaries in 1927 brought forth the sterling qualities of Mr. Schoenberger, upon whose shoulders fell a large share of the responsibility for the protection of the levees and property of thousands of people in the Lower Valley. He took a leading part in the cutting of the Caernarvon Levee below New Orleans, creating the artificial crevasse which was to relieve the danger of a break at the city above. Supervising the holding of the levees in the upper section of the State occupied every available hour of his time, and he fought to the last ditch for every foot of levee. It was only after everything that was humanly possible had been done that the several disastrous crevasses occurred. It was a losing fight at the time, but with defeat came victory, for the great disaster opened the eyes of the entire country to the need of adequate flood control.

With complete victory in sight, his quarter-century battle against the flood menace of the Mississippi River came to an end on Monday, September 24, 1928, when he died suddenly from a heart attack at his home. Although complaining of pains in the region of the heart, he had arisen as usual, dressed, and prepared to go to his office. While in the garden of his home, he was stricken and died before the arrival of a physician. Mr. Schoenberger was described as a martyr to flood-control work by his physician who pronounced the weak condition of his heart as due to this strenuous work during the fight to save the levees in 1927, and his labors in Washington, D. C., in behalf of Federal flood control.

Public tribute was paid to Mr. Schoenberger after his death by his numerous friends and admirers, many of whom held high office. The following editorial published in New Orleans, just after his death, expresses the esteem in which he was publicly held:

"Personally modest and unassuming, George C. Schoenberger was one of those talented Louisianians whose work had escaped general public notice until last year when the greatest flood in the history of the Mississippi River centered attention on the men who bore the brunt of the battle against almost hopeless odds. In this crisis he emerged as an effective and skillful leader of the State's flood forces, whose command he had assumed two years before upon his appointment as Chief of State Engineers.

"Although disaster after disaster overwhelmed the flood-plain of the State, his courage never slackened, nor was there ever any flagging in his resolution to fight to the last to save every possible acre from destruction and ruin. His personal presence was felt wherever the water threatened in the far-flung battle line from the south bank of the Arkansas River to the Gulf of Mexico. In the long history of Mississippi River floods probably no more critical situation could be cited than that existing on the memorable night of Good Friday, 1927, when a rain and windstorm whipped the waters of the swollen Mississippi between New Orleans and Baton Rouge, shattering levees at a dozen different points, and threatening imminent disaster, which was averted only after sev-

eral days of furious work. The holding of the levee lines on the Mississippi below Red River Landing until the Bayou des Glaises crevasses had relieved the pressure in the main stream, was a notable achievement in itself.

"For the task which confronted him in the 1927 flood Mr. Schoenberger had been in training since his graduation from the State University in 1898, practically his entire time since then having been devoted to river work. He had espoused the spillway cause along with other notable engineers prior to the 1927 flood, and in the fight at Washington last fall and winter for adequate flood control legislation his views had marked effect. His unexpected death, following a year and a half of intense activity, first in the gruelling battle to hold the levees and afterwards at Washington, is a distinct loss to the State, to the service of which he had devoted the most useful years of his life."

He was married on January 8, 1902, to Podine Conrad Pope, of Baton Rouge, La. He is survived by his widow, three sons, George C., Jr., Ripley Pope, and Sidney Conrad, and one daughter, Podine.

Mr. Schoenberger was elected an Associate Member of the American Society of Civil Engineers on November 27, 1917.

WALTER TUTTLE SPENCER, Assoc. M. Am. Soc. C. E.*

DIED AUGUST 19, 1928.

Walter Tuttle Spencer was born in Guilford, Conn., on July 5, 1869, the youngest of three sons of Christopher Spencer and his wife, Jane E. Collins Spencer. He prepared for college at Guilford Institute, and entered the Sheffield Scientific School of Yale University from which he was graduated in 1890.

Immediately after his graduation, Mr. Spencer entered the service of the New York, New Haven and Hartford Railroad Company, with which Company he was connected for thirty-eight years. From 1893 to 1914, he was in the Engineering Department, having been appointed Division Engineer of the Northampton-Air Line Division in 1893. From this time he served, successively, as Division Engineer on the Highland, Hartford, Western, and Providence Divisions. In 1914 he was transferred to the Operating Department, as Superintendent of the Old Colony Division at Taunton, Mass.; and, in 1919, he was appointed Superintendent of the Boston Division with headquarters at Boston, Mass., which position he held at the time of his death.

Mr. Spencer's abilities as a Railroad Engineer were of the highest quality. He handled many problems of railroad construction, as well as maintenance work successfully and satisfactorily. Some of the more important of these were: The double-tracking of the Air Line Division; and of the Western Division; the Morgan Street Yard Improvements, at Hartford, Conn.; elimination of grade-crossings, at East Hartford; the Walnut Street improvements, at Hartford; elimination of grade-crossings at Westfield and Northampton, Mass.; the Farmington River Bridges, at Farmington and Collinsville, Conn.; the first concrete arch bridge ever built by the "New Haven", located at West Haven, Conn.; the Park River Bridge, at Hartford; and numerous grade-crossing elimination projects which the "New Haven" Railroad was developing during Mr. Spencer's service as Division Engineer.

His later position as Superintendent of a Division gave him a varied experience in the three Southern New England States. The requirements of his calling made it necessary for him, during periods of severe weather, or heavy traffic, to remain at his desk for days at a time. In his handling of men he believed firmly in the principle of even-handed justice. Where the matter of discipline was involved he would listen with patience to the plea of one for the shortcomings of another and he always tempered justice with mercy. It was as if the person in difficulty were some one close to him.

Mr. Spencer was married to Dorothy E. Dain, of Waterbury, Conn., who survives him, as does one brother, Frederick C. Spencer, of Guilford, Conn.

He was a member of the Connecticut Society of Civil Engineers, the Traffic Club of New England, the New England Railroad Club, and the New Haven Railroad Club.

* Memoir prepared by Paul Sterling, M. Am. Soc. C. E.

Walter Tuttle Spencer was a modest man. He was never in the forefront, always retiring and reticent, always even-tempered and soft-spoken. To know him was to love him. The high place which he held among his associates had been won by his own ability and single-hearted devotion. For him, life ended on a high plane.

Mr. Spencer was elected a Junior of the American Society of Civil Engineers on May 2, 1893, and an Associate Member on October 4, 1899.

FRANK ANDREW TILLMAN, Assoc. M. Am. Soc. C. E.*

DIED SEPTEMBER 2, 1927.

Frank Andrew Tillman, the son of Peter M. and Ida Tillman, was born on September 27, 1888, at Waverly, N. Y. His early education was obtained in the Waverly Public Schools, after which he attended the University of Colorado at Denver, Colo., for two terms from October, 1908, to June, 1909, and from September, 1909, to June, 1910, taking special night and part-day studies in mathematics, plain surveying, municipal and irrigation engineering, construction, and design.

During the summers of 1908 and 1909, Mr. Tillman was engaged as Chainman, Rodman, and Topographer on preliminary and location surveys for the Denver, Laramie, and Northwestern Railroad, and as Rodman on Construction.

In June, 1910, he accepted a position as Transitman in charge of party on a land survey of 65 000 acres, comprising section line, irrigation ditch, fence line, building location, and general development, near Laramie, Wyo. He also spent some months on coal mine surveys at Medicine Bow, Wyo., and on the general layout of the Town of Milliken, Colo. In the fall of 1911 he acted as Levelman, for the Goldsborough Engineering Company, of Chicago, Ill., on a topographical survey of 30 000 acres in Wyoming, later, becoming Draftsman and Computer for this Company in Denver, Colo., on irrigation structures, lateral and canal projects, etc.

From January to April, 1913, Mr. Tillman was employed as Office Engineer for the Denver Realty Company on sub-divisions and utilities. He then returned to the Goldsborough Engineering Company as Chief of Construction on irrigation structures, canals, and laterals, until September, 1913, when he was engaged as Engineer for the Enalpac Oil and Gas Company on the design, location, and construction of pump-houses, refineries, derricks, and pipe lines, at Casper and Cody, Wyo.

In July, 1915, he came East to Binghamton, N. Y., where he was employed until May, 1917, by the City as Instrumentman on preliminary surveys and location for an intercepting sewer and other municipal work. He subsequently became Assistant City Engineer for the City of Binghamton in charge of sewer construction and design.

In September, 1918, Mr. Tillman entered the United States Government Service at Perth Amboy, N. J., on housing projects, topography, utilities, and street planning. The following year, however, he returned to Binghamton, where he acted as Secretary to the Commissioner of Public Works and Superintendent of Parks.

In 1920, he accepted a position as Assistant Engineer with Hoadley and Giles, a civil engineering firm of Binghamton, in charge of field parties and on general engineering work. He assisted in the design of the water supply and sewer systems of Johnson City, N. Y., and designed the proposed water system for Endicott and West Endicott, N. Y. He also assisted in the sur-

* Memoir prepared by John A. Giles, M. Am. Soc. C. E.

veying and planning of the various subdivisions, and served as Office Engineer on computations and design.

In April, 1921, Mr. Tillman was appointed Commissioner of Public Works and Engineer of Johnson City, in charge of the design, construction, and maintenance of all public utilities. He held this position until his sudden death on September 2, 1927, from intestinal pneumonia.

In 1921, he was married to Lois A. Kilmer, of Binghamton, who, with a daughter, survives him.

Mr. Tillman was a member of the Baptist Church and prominent in the Masonic Fraternity as a member of Waverly Lodge, A. F. and A. M., the Scottish Rite Bodies, of Cheyenne, Wyo., and Kalurah Temple, Nobles of the Mystic Shrine, of Binghamton, N. Y. He was also a member of the Kiwanis Club of Binghamton, N. Y.

He was most popular among his many friends and associates in the Engineering Profession, all of whom liked him for his genial disposition and respected him for his ability and thoroughness as an engineer. As a public official, he incurred the good-will of his associates and commanded the respect of the public.

Mr. Tillman was elected an Associate Member of the American Society of Civil Engineers on May 28, 1923.

EDWARD JOHN TULLY, Assoc. M. Am. Soc. C. E.*

DIED OCTOBER 27, 1928.

Edward John Tully was born at Omaha, Nebr., on June 23, 1893, the son of John Charles Tully and Gertrude (Stein) Tully. Most of his life was spent in Omaha, where he attended the public schools, finishing High School in 1909.

He was employed by the Concrete Engineering Company, in Omaha, for several years, leaving to enter the Government service during the World War. During this period he was engaged in the Ship Building Department, first in Washington, D. C., and then in Philadelphia, Pa., later going to Camp Funston, Kansas. He was not sent overseas, however.

After returning to Omaha, Mr. Tully was employed by the Metropolitan Utilities District. He was Chief Draftsman for the design of the Omaha Filter Plant built in 1923 at a cost of $600 000 for improving the water supply of the City. As Draftsman, he also assisted in the design of a 48-in. water main, 7 miles long, costing in excess of $800 000. He designed the foundation and connections for two pumps with capacities of 30 000 000 gal. per day. After serving with the Metropolitan Utilities District for three and one-half years, under the supervision of C. D. Robinson, M. Am. Soc. C. E., Mr. Tully was forced to resign on account of illness.

In an unsuccessful attempt to regain his health, he spent the following two years, until his death, in California. When Mr. Tully left Omaha for Sierra Madre, Calif., he was engaged to be married to Genevieve Kennedy a sweetheart of several years. On account of his illness they thought it advisable to delay the marriage. On the advice of relatives, however, Miss Kennedy accompanied by Mr. Tully's aunt, went to California, arriving on a Tuesday. The wedding ceremony was performed Wednesday afternoon and Mr. Tully passed away Saturday morning. Mr. Tully's remains were brought back to Omaha from California, and he was laid to rest in St. Mary's Cemetery, beside his mother and father.

Mr. Tully was very fond of outdoor sports. He was an expert swimmer and skater, and played baseball on several junior teams. He was very ambitious, possessing a keen, creative mind. During his school days he worked in his uncle's grocery store from five o'clock in the morning until it was time for him to go to school, and then again after school hours and on Saturdays. He also attended night school to further his engineering knowledge, and because he loved his work, was always studying during his sickness. He associated with older men rather than men of his own age, having such interest in his work that he felt they could help and advise him from experience. A man of likeable personality, he was quiet and reserved, but loved by all who knew him. Had he lived longer, he doubtless would have been very successful. Among the associations that he cherished most, were his connections with the Society.

Mr. Tully was elected an Associate Member of the American Society of Civil Engineers on August 30, 1926.

* Memoir prepared by Mrs. Genevieve Kennedy Tully, Omaha, Nebr.

ALBERT ALEXANDER VAN PETTEN, Assoc. M. Am. Soc. C. E.*

DIED NOVEMBER 27, 1927.

Albert Alexander Van Petten was born at Farmington, Ill., on June 9, 1883. He was graduated from the University of Illinois in 1907 with the degree of Bachelor of Science. While he was still attending college, he did apprentice work in civil engineering with the Illinois Central Railroad Company, the Yazoo and Mississippi Valley Railroad Company, and the South Porto Rico Sugar Company.

After graduation, Mr. Van Petten re-entered the employ of this latter Company and, in 1907-1909, had charge of surveying and mapping cane lands, as well as the planning of a fire protection system, including a 3 000 000-gal. reservoir, for the Company's sugar factory. He also made surveys and plans for an irrigation pumping plant furnishing water to 1 000 acres and planned and installed a gravity irrigation system covering 1 500 acres of land. Later, for the same Company, he took part in the planning and construction of three irrigation pumping plants and the re-construction of a 2 500-ton sugar factory on the Fortuna property of the Company near Ponce, Porto Rico.

From Fortuna, he was transferred to the Pagan property of the Company, where he designed and constructed railroad yards for the transference of cane, and completed surveys for the installation of three irrigation pumping plants, each of a daily capacity of 4 000 000 gal. In addition, he surveyed and completed plans for a diversion dam to supply water for a gravity irrigation system.

From 1912 to 1914, inclusive, Mr. Van Petten was Superintendent of Construction of the Dam and Power House at Comerio, Porto Rico, for the San Juan Light and Power Company. He completed this work and from it is derived the power for the electric lighting and street railway system of San Juan, Porto Rico.

During the early part of 1915 he entered the service, as Chief Civil Engineer, of the Punta Alegre Sugar Company, which was then engaged in the development of a large sugar property at Punta San Juan, Cuba. This project included the planting of about 16 000 acres in cane, the construction of 50 miles of standard gauge railroad, the erection of a sugar factory with a grinding capacity of 3 000 tons of cane per day, and the construction of living houses for several hundred employees.

During the progress of this development work, a serious drainage problem arose, threatening, through the inundation of about one-half the cane area, to jeopardize the whole investment valued at several million dollars. Lands of higher elevation, beyond the boundaries of the Company's holdings, delivered their drainage water on these cane lands and kept them submerged long enough to effect serious damage to the crop. Mr. Van Petten met this situation by constructing a diversion drainage ditch at the highest elevation of the Company's land, which completely prevented further inundations and safeguarded the entire investment. Before the completion of this project by the

* Memoir prepared by H. Edson, Esq., New York, N. Y.

Punta Alegre Sugar Company, Mr. Van Petten was made its General Manager and, as such, brought the property into operating condition.

In 1919, he took service as Civil Engineer with the Barahona Sugar Company, which was engaged at that time in developing a sugar property near Barahona, Santo Domingo. This project embraced the clearing of 20 000 acres of land, and planting them in cane; the planning and construction of a complete irrigation system covering the entire area and including the construction of a diversion dam in the Yaque del Sur River, a stream subject to torrential freshets, the laying out and construction of 60 km. of meter gauge railroad; the erection of a sugar factory with a daily capacity of 4 000 tons of cane; the construction of docks for loading cargo boats; the building of villages to house about 5 000 people; the construction of an aqueduct, 13 km. long to supply these people with fresh water as well as the people of the adjoining Town of Barahona, with about the same population; and, finally, the planning and construction of a drainage system to carry off the surplus irrigation water from the canefields.

On this Barahona work, as was the case in the Punta Alegre development, Mr. Van Petten's ability, both as a civil engineer and as an executive, became so apparent that he was made General Manager of the whole project and, as such, successfully completed it and turned it over to the operating company in 1922.

At the completion of his Barahona work, Mr. Van Petten was connected with the Macareño and General Sugars Companies in the development of their sugar properties in Cuba and finally accepted the position of General Manager of the Mercedes property belonging to the Cuba Cane Sugar Corporation, where he remained until his death on November 27, 1927.

On November 6, 1907, Mr. Van Petten was married to Jessye Gurney, who survives him, with four children, one son, Douglas, and three daughters, Willa, Alberta, and Jeanne.

Mr. Van Petten was a Mason and a member of the Island Chapter, Havana, Cuba.

His sterling qualities endeared him to all with whom he came in contact and his ability marked him as one of the outstanding construction and operating men of the sugar industry, with which he had been connected during the greater part of his active life. He was one of those singularly attractive and dependable men whose death is an irreparable loss to his family. He leaves a void which his friends find it impossible to fill.

Mr. Van Petten was elected an Associate Member of the American Society of Civil Engineers on December 5, 1911.

HOWARD ELLIOTT, Affiliate, Am. Soc. C. E.*

DIED JULY 8, 1928.

Howard Elliott was born in New York, N. Y. on December 6, 1860, the son of Charles Wyllys Elliott and Mary (Whyte) Elliott. Charles Elliott Perkins, who as President so efficiently guided the destinies of the Chicago, Burlington and Quincy Railroad Company during its period of greatest expansion, was his cousin.

Mr. Elliott's early inclinations are shown in the fact that after passing through the High School of Cambridge, Mass., he entered what was then known as the Lawrence Scientific School of Harvard University, from which he was graduated with the degree of Civil Engineer in 1881. He plunged at once into railroad work, and, in October of that year, he entered the employ of the Burlington Railroad Company, with which he was to be intimately connected, either directly or indirectly, for many years. As a matter of fact he had been a Rodman on that road the year before he received his degree.

While he served in various capacities on various lines during his lifetime, Mr. Elliott's specific service as related to engineering may be said to have begun in 1896, when for five years he was General Manager of the Missouri Lines of the Burlington Railroad. In 1901, he became Second Vice-President of that Company, from which position, in 1906, he was promoted to the Presidency of the Northern Pacific Railway Company. He made his home at this time in St. Paul, Minn. From 1903 to 1913 there was a very marked development in the Northwest. During his administration as President he directed the greatest construction program ever attempted by the Northern Pacific Railway. The capacity of the road was greatly increased, and the agricultural districts were better served by the construction of branch lines, and, in addition, the West Coast District was improved between Portland, Ore., and the Canadian boundary.

The late James J. Hill, F. Am. Soc. C. E., was then the dominant figure in the Northwest, especially in the group of railroads comprising the Great Northern, Northern Pacific, and Burlington Companies. The late E. H. Harriman was the leader in the Union Pacific and Southern Pacific group. It was Mr. Elliott's good fortune to be associated with these remarkable men and to take part in the memorable "Hill-Harriman" contests in railroad building in Washington, Idaho, and Oregon. This was a most important period in the history of the Northern transcontinental lines. While some remnants of the old feeling of bitterness remained, the community of ownership which had come about as a result in the formation of the Northern Securities Company was established more strongly day by day. Thus, the Great Northern and Northern Pacific Systems, growing more closely together, presented a united front to the rather insistent demands of the Harriman System. As part of the complete fortification, in a railroad sense, of the Pacific North-

* Memoir prepared by Ralph Budd and W. L. Darling, Members, Am. Soc. C. E.

west, the Spokane, Portland and Seattle Railway was constructed from Spokane, Wash., to Portland, a distance of 380 miles. Mr. Elliott had his part in this joint project, the conception and responsibility for which, of course, belonged directly to Mr. Hill.

The Northern Securities Company was declared illegal, under the Sherman Anti-Trust Law, upon suit by the United States Government; but the vital feature of this decision was that it did not affect the joint ownership established by the two Northern Lines of the Burlington property, its stock having been purchased outright and bonds issued to finance the purchase. At this time there was a deep hostility throughout the United States to railroad policies, and this enmity had to be turned first into neutrality and then into friendship. To succeed in this plan it was necessary systematically to carry on public education in the fundamentals of the situation. As a result Mr. Elliott was an important factor in such work through all his active life. He traveled across the Continent many times and made scores of addresses (some of which were printed), in which facts about the railroads were dispassionately presented before commercial clubs, business men's associations, and other bodies. The more formal talks which were distributed widely in the Northwest no doubt had a wholesome effect in the territory served by the Northern Pacific System, and were very helpful in directing the attention of the people to the need of remedial legislation.

By the summer of 1913 there had developed in New England a railroad situation as unpromising and as difficult to handle as that which had existed in the territory of the two Northern transcontinentals. The New York, New Haven and Hartford Railroad Company was the great railroad corporation of New England. It had been expanded to take in most of the railroad properties of that section, and its finances had become seriously affected. Much hostility had been aroused against the management of the road, particularly over the policy which included a consolidation of all New England transportation activities—steam railroads, trolleys, and steamship companies—to be dominated by the New Haven Company. A number of prominent New Englanders interested in an adequate and permanent transportation service for New England, chief among them Mr. Elliott's friends, the late Henry Lee Higginson and Theodore N. Vail, invited and urged Mr. Elliott to come East and take up the task. As his children were either at school or college in the East, and practically all his relatives and old friends were there, he found the idea attractive. Then, too, since the Board of Directors of the Northern Pacific Company met in New York, it had been necessary for him to make from eight to ten trips a year between St. Paul, Minn., and the Atlantic Coast, which, with the necessary inspection trips in the West as far as the Pacific Coast, represented traveling from 30 000 to 40 000 miles per year.

Mr. Elliott, realizing to some extent the seriousness of the situation, decided to take up the work, after discussing the matter fully with friends in Boston, Mass., New York, and Philadelphia, Pa. Making his residence in Boston, he became associated with the New York, New Haven and Hartford Railroad Company as its President on September 1, 1913, and soon became Chair-

man of the Board of Directors, being succeeded by Mr. James H. Hustis as President. At that time the New Haven System consisted not only of its rail lines, but also included in its ownership or control the Central New England Railway Company, the Boston and Maine Railroad Company, the New York, Ontario, and Western Railway Company, the New England Steamship Company, the Hartford and New York Transportation Company, substantially all the street railway lines in Rhode Island and Connecticut, and the important street railways of Massachusetts, in Berkshire County, and in the Cities of Springfield and Worcester. It was expected that the Chairman of the System would exercise general oversight of the holdings of the New Haven Company, each of which was to be in charge of a President, and that the Board of Directors of the New Haven Lines, being responsible for the parent company and its holdings, should act as a supreme advisory body to the Chairman. These plans were soon upset by the action of the Federal Government, however, in bringing suit to compel the New Haven Company to give up the Boston and Maine Railroad, and the Massachusetts, Rhode Island, and Connecticut trolleys.

Mr. Elliott labored long and earnestly to avert the Consent Decree, which was forced on the New Haven Company by the Administration at a time when this Company, because of its financial embarrassment, could not afford to resist the Government. Mr. Elliott accepted the situation, however, and with what remained of the New Haven System began earnestly to present to Southern New England the serious problems which confronted the transportation agency and the communities which were dependent upon it. In his speeches and writings he urged the necessity for confidence, patience, and co-operation, and, greatly to his credit, secured them all. His attitude and action inspired a revival of the belief that the New Haven System might be restored to its former high state. Incessant work in this field impaired his health, and on May 1, 1917, he resigned as Chairman and President, and took a position on the Railroads' War Board, at Washington, D. C., where, together with the other members of the Board, he represented all the railroads until the beginning of Federal Control in the same year.

In 1920, Mr. Elliott became Chairman of the Board of Directors of the Northern Pacific Railway Company, and took an active part in the management of that property until his death. He was also a Director of the Chicago, Burlington and Quincy Railroad Company from 1918 until his death, as well as a Director of many companies connected with railroads and other allied interests, as well as a member of many clubs and scientific, business, and social organizations.

Mr. Elliott was also a very ardent and enthusiastic Harvard man, and Harvard University, in turn, thought most highly of him. As early as 1909 he was elected one of its Board of Overseers, and the Alumni Association made him its President for the year 1917. He was extremely interested in the work of the University, especially that of the School of Transportation, founded in honor of Mr. James J. Hill.

The honorary degree of Doctor of Laws was conferred on Mr. Elliott by several colleges, namely, Middlebury College, Vermont, in 1916; Trinity Col-

lege, Connecticut, in 1914; and Carleton College, Minnesota, in 1927. He also received the honorary degree of Doctor of Science from the University of Pennsylvania in 1927.

Mr. Elliott was married on October 12, 1892, to Janet Churchill. They had two daughters and one son. He was an extremely attractive man personally because he seemed actually to radiate kindness and good will. These qualities contributed greatly to his practical success.

Mr. Elliott died on July 8, 1928, while still actively engaged in the handling of railroad affairs to which so much of his life had been devoted. Few men have left behind them a larger circle of appreciative associates and admiring friends.

Mr. Elliott was elected an Affiliate of the American Society of Civil Engineers on June 5, 1900.

WILLIAM PENDLETON PALMER, Affiliate, Am. Soc. C. E.*

DIED DECEMBER 17, 1927.

William Pendleton Palmer, the son of James Stewart and Eleanor Pendleton (Mason) Palmer, was born in Pittsburgh, Pa., on June 17, 1861. His father was a Captain in the 155th Regiment of Pennsylvania and died during the Civil War. His mother was a descendant of an old Colonial family.

After completing a four years' course which included Mechanical Drafting, he was graduated from the Pittsburgh Central High School in 1878. Subsequently, he entered the employ of the Lewis, Oliver and Phillips Company, with which he remained until May, 1881, occupying various minor positions, as well as that of Traveling Salesman.

On May 27, 1881, Mr. Palmer began his service with Wilson, Walker, and Company, which, on January 1, 1886, was absorbed by Carnegie, Phipps, and Company, Limited. In July, 1892, the latter Company was united with the Carnegie Steel Company, Limited, and, with these various Carnegie interests, Mr. Palmer occupied, successively, the positions of Draftsman, Traveling Salesman, Secretary, General Sales Agent, and Assistant to the President.

In 1896, he resigned from the latter position to accept an appointment as Second Vice-President of the Illinois Steel Company, in Chicago, Ill. When the American Steel and Wire Company of New Jersey was organized, on January 13, 1899, he became a Director and General Manager of that Company. On May 9, 1900, Mr. Palmer was elected President of the American Steel and Wire Company and held this position until his death.

Mr. Palmer was also President of the American Mining Company. He was made a Director of the United States Steel Corporation in 1922, and was re-elected in 1924 and 1927. He likewise acted as a Director of the H. C. Frick Coke Company and of the Cleveland Trust Company, and was a Trustee of the Lakeside Hospital, Case Library, and Oberlin College. He was a member of the Protestant Episcopal Church.

Mr. Palmer had a predilection for historical literature. He gathered from various sources all that is known of the wire industry. This information, in the form of written data, together with exhibits of machinery used in the industry, forms a notable collection known as the Industrial Museum of the American Steel and Wire Company, located at Worcester, Mass.

It was also Mr. Palmer's pleasure to collect historical data and Americana, particularly books, maps, manuscripts, photographs, etc., both Union and Confederate, relating to the Civil War, Abraham Lincoln, and slavery. This collection, believed to be the largest in the country outside the Library of Congress, he presented to the Western Reserve Historical Society of Cleveland.

Mr. Palmer was married on July 24, 1898, to Mary Boleyn Adams, of Chicago, who, with a daughter, Mrs. William Chisholm, II, and a son, William Pendleton, Jr., survives him.

He had served as President of the Western Reserve Historical Society since 1913 and was a member of the following societies and clubs: The Engineers

* Memoir prepared by George H. Tinker, M. Am. Soc. C. E.

Society of Western Pennsylvania; the Cleveland Chamber of Commerce; The Chamber of Commerce of the United States; the Society of Colonial Wars; the American Antiquarian Society; the Military Order of the Loyal Legion; the Union Club; the Rowfant Club; the Cleveland Country Club; the Tavern Club; the Mayfield Club; the Kirtland Club; and the Chagrin Valley Hunt Club of Cleveland; the Duquesne Club and the Pittsburgh Club, of Pittsburgh, Pa.; the Chicago Club, of Chicago, Ill.; and the Engineers' Club and India House, of New York, N. Y.; also the Worcester Club and the Rufus Putnam Society, of Worcester, Mass.; and the Megantic Fish and Game Corporation of Belmont, Mass.

Mr. Palmer was elected an Affiliate of the American Society of Civil Engineers on March 3, 1896.

CHARLES FREDERICK QUINCY, Affiliate, Am. Soc. C. E.*

DIED OCTOBER 1, 1927.

Charles Frederick Quincy, the son of George Henry and Mary Caroline (Sweetser) Quincy, was born in Newton, Mass., on July 16, 1856. On his father's side he was a direct descendant of Edmund Quincy who came from England in 1628 and settled in Massachusetts.

Mr. Quincy's elementary training was received at the Newton High School which, together with some time spent under the guidance of a private tutor, completed his education. It was during this period that he spent two years at sea, due to ill health, and was shipwrecked off the coast of Denmark.

In 1886, he entered the railroad supply business, a founder of the firm which first operated as the Dunham Manufacturing Company and, later, as the Q and C Company, of which he became President in 1887, and with which he was connected during the remainder of his active business life. Subsequent to 1922, Mr. Quincy gradually lessened his active participation in the work of the Company, and, in 1926, he was appointed Chairman of the Board of Directors, which office he held until his death. Mr. Quincy passed away on October 1, 1927, at his summer home, Centre Harbor, N. H.

He was also Chairman of the Board of the Dorr-Miller Differential Company, and the Miller Transmission Company and Vice-President and Director of the Kelvinator and Westchester Company.

Mr. Quincy was a member of the American Society of Mechanical Engineers, American Institute of Mining and Metallurgical Engineers, American Association for the Advancement of Science, and the American Society for Testing Materials.

He also held membership in the Railroad Club, the Greenwich Country Club, and the Lotos Club of New York, the Chicago Club, the New York Botanical Society, the New York Zoological Society, the National Geographical Society, and the Metropolitan Museum of Natural History.

Mr. Quincy's entry into the various mechanical societies was due to his success in presenting new railroad devices to the railroad trade. He was a great friend of the under-privileged boy, having been the first President of the Allendale Association at Lake Villa, Ill., and a member of the Board of the Boys Club Federation of New York and Hope Farm, at Verbank, N. Y.

On October 22, 1879, Mr. Quincy was married to Etta Molineaux Ives. He is survived by his widow and four children, Mrs. A. Q. Karcher, and Polly, Edmund, and Roger Bradshaw Quincy.

Mr. Quincy was elected an Affiliate of the American Society of Civil Engineers on February 4, 1896.

* Memoir compiled from information on file at the Headquarters of the Society.

LUCERN S. STEWART, Affiliate, Am. Soc. C. E.*

DIED NOVEMBER 29, 1927.

Lucern S. Stewart was born at Marshall, Ill., on May 5, 1856. He received his education in the public schools of Illinois.

Mr. Stewart's first bridge work was with the Wabash, Chester, and Western Railroad Company, in Illinois, and he was rapidly promoted to be Superintendent of Bridges.

In 1885, he was employed by the late George S. Morison, Past-President, Am. Soc. C. E., Chief Engineer, to build the caissons for the Illinois Central Railroad Bridge over the Ohio River, at Cairo, Ill. After finishing this work, Mr. Morison sent him to the Merchants Bridge, in St. Louis, Mo., then under construction. He was soon transferred in 1887, to Memphis, Tenn., and made General Foreman of Substructure on the Kansas City, Fort Scott and Memphis Railroad Bridge over the Mississippi River. The construction of this bridge extended over a period of about five years, and the experience obtained under Mr. Morison as Chief Engineer, and the late Alfred Noble, Past-President, Am. Soc. C. E., Resident Engineer, was invaluable to Mr. Stewart. When the Memphis Bridge was finished in 1892, he took charge of the substructure work on the Burlington Bridge over the Mississippi River, near St. Louis.

In 1893, Mr. Stewart entered the service of the late A. J. Tullock, M. Am. Soc. C. E., Proprietor of the Missouri Valley Bridge and Iron Works, as Superintendent on the substructure of the Missouri River Bridge, at Leavenworth, Kans., and in 1895 and 1896, he was Superintendent of the Missouri River Bridge, at Jefferson City, Mo.

In 1896, when contract work was almost at a standstill because of a financial crisis, Mr. Stewart was employed by the Kansas City, Fort Scott and Memphis Railroad Company, for which he carried on an extensive reconstruction program on bridges until August, 1898, when he again entered the service of Mr. Tullock. During the next six years he had charge of the following large substructure contracts: Reconstruction of some of the old piers on the St. Louis, Iron Mountain, and Southern Railway Bridge over the Arkansas River, at Little Rock, Ark.; the Kansas City, Pittsburgh and Gulf Railway Bridge over the Red River, at Index, Tex.; the Tampico Wharf for the Mexican Government at Tampico, Mexico; the Texas and Pacific Railway Bridge over Old River at Steamboat Island, in Louisiana; the El Paso and Southwestern Railway Bridge over the Rio Grande, near El Paso, Tex.; the Fort Smith and Western Railway Bridge over the South Canadian River, at Garner, Indian Territory, and two bridges for the Memphis, Helena and Louisiana Railway Company (Missouri Pacific Lines) over the Arkansas and White Rivers, in Arkansas.

* Memoir prepared by William Harris Bosler, M. Am. Soc. C. E.

After the death of Mr. Tullock in 1904, the Missouri Valley Bridge and Iron Company was organized, and Mr. Stewart was made Superintendent of Construction, which position he held until December, 1906, when he resigned to organize the Union Bridge and Construction Company, of which he was made President. He continued as President of this Company until October, 1921, when he was made Chairman of the Board of Directors, which position he retained until his death.

After the organization of his own company, Mr. Stewart successfully executed a number of large substructure contracts, some of which were: The Mississippi River Bridge, at Keithburg, Ill.; two bridges at Portland, Ore., over the Willamette River; the Harahan Bridge over the Mississippi River, at Memphis, Tenn.; the Paducah and Illinois (Burlington) Bridge over the Ohio River, at Metropolis, Ill.; the New Orleans and Pacific Railway Bridge over the Atchafalaya River, in Louisiana; and two bridges for the Northern Pacific Railway Company, over the Yellowstone and Missouri Rivers, in Montana.

In 1917, when the United States Government called for contractors to build ships, Mr. Stewart accepted contracts to build six 3 500-ton hulls of the Ferris type, three cargo hulls of the 2 000-ton type, and six 5 000-ton hulls of the Daugherty type. The first nine of these hulls were successfully completed and went into service, but those of the Daugherty type were never completed, the contract having been cancelled after the signing of the Armistice.

Practically all the substructure work which Mr. Stewart executed, was done by the pneumatic process. He probably acquired more practical experience and a greater knowledge of pneumatic foundation work than any other man of his time. The base on which this experience and knowledge was founded, was ability, integrity, energy, and resourcefulness, combined with a keen foresight, and the faculty to judge men.

As evidence of the esteem in which he was held by engineers, the following is cited: In 1902, a contractor of New York City inquired of Mr. Alfred Noble the name of the best foundation man in the United States to take charge of an important piece of work. Mr. Noble replied "the name of that man is L. S. Stewart, and you will be fortunate if you can secure his services".

From Ralph Modjeski, M. Am. Soc. C. E., comes the following tribute:

"I have known Mr. Stewart for many years; my first acquaintance with him was about 36 years ago, when, as general foreman, he was in charge of building the difficult foundations of the old Memphis Bridge. Later, as head of the Union Bridge and Construction Company, he built the foundations and piers of the second Memphis or Harahan Bridge. On these important pieces of work, as well as on many others, he commanded the admiration, respect, and friendship of every one who came in contact with him. Every one recognized in him great energy, resourcefulness, thoroughness, and absolute integrity. It was often said that 'no one could knock the caisson together quicker and better than L. S. Stewart'. One felt that within his cheerful, amiable and even disposition, dwelt an indomitable energy and honesty of purpose. Through his death the Engineering Profession and his many friends have suffered an inestimable loss."

In 1923, Mr. Stewart practically retired from active service, and moved with his family to Tampa, Fla., where he remained until his death. He is survived by his widow and one son, Mr. O. H. Stewart, of Tampa.

He was a York Rite Mason and a member of Abdallah Temple, A. A. O. N. M. S., of Leavenworth, Kans.

Mr. Stewart was elected an Affiliate of the American Society of Civil Engineers on February 6, 1912.

MARIO JOHN MIDOLO, Jun. Am. Soc. C. E.*

DIED MARCH 11, 1926.

Mario John Midolo, the son of John and Kathryn Midolo, was born in Syracuse, Italy, on May 21, 1900, and received his primary education in his native country. When the boy was seven years old, the family came to America and settled in New York, N. Y., where he continued his elementary education in the public schools. This at first proved quite difficult as he was not well versed in the English language; but it was not long before he was one of the most promising students in the school he attended.

After remaining in New York about five years, the family removed to Philadelphia, Pa., where Mr. Midolo entered the Drexel (Public) School, and finished his elementary course in 1915. At this time, the question of higher education and his future profession was uppermost in his mind. He then entered the South Philadelphia Manual Training High School and finished the course there which brought him nearer his goal, namely, the profession of Civil Engineering. He was graduated well up in his class in February, 1919, and, owing to the record that he had made while a student, he was chosen to enter Pennsylvania State College, in September, 1919.

This proved to be another stepping stone toward his ambition, and his untiring efforts and application were soon rewarded. He was graduated from Pennsylvania State College in June, 1923, with the degree of Bachelor of Science in Civil Engineering.

Engaging in his profession, Mr. Midolo first served as Assistant to the Superintendent on Sanitary Sewer Construction for the Perna Construction Company. Later, he followed Land Surveying and was associated with Mr. Thornton Hollingshead, of Morristown, N. J. Shortly afterward he accepted a position as Field Engineer with the Barber Asphalt Paving Company, of Philadelphia.

As he had now had considerable professional experience, he felt he would like to make a permanent connection. Accordingly, Mr. Midolo took the civil service examination in connection with the Bureau of Surveys, City of Philadelphia, and was successful in securing an appointment in one of the District Offices (7th Survey District), where he remained until September 15, 1925. He then accepted a position as Engineer and Estimator for Mr. Frank Brennan, a well known private contractor in South Philadelphia.

It was while in Mr. Brennan's employ that Mr. Midolo contracted his fatal illness. Shortly before Christmas, 1925, he caught a heavy cold and was compelled to remain at home for a time, but his energy and love for his work took him back to the office. Although ambitious to continue his work, his health gradually began to decline, and he was compelled to resign his position.

* Memoir prepared by Francis A. Cotney, Esq., Philadelphia, Pa.

He died on March 11, 1926, and is survived by his father and several brothers and sisters, all residents of Philadelphia. Modest and unassuming, and devoted to his family, he was cut off at the beginning of a promising career.

Mr. Midolo was elected a Junior of the American Society of Civil Engineers on December 15, 1924. He was also a member of the Penn State Club of Philadelphia.

WILLIAM BACON WELLS, Jun. Am. Soc. C. E.*

DIED FEBRUARY 17, 1928.

William Bacon Wells, the son of George William Wells and Amy Bacon Wells, was born at Lewis, Kans., on August 26, 1899. He received his Grammar and High School education at Lewis, and studied Civil Engineering at Kansas University, from which he was graduated in 1922 with the degree of Bachelor of Science in Civil Engineering.

After his graduation, Mr. Wells was engaged as Assistant City Engineer in Ellsworth, Kans. In this capacity, he acted as Resident Engineer on the construction of the water-works system in that city.

In 1923 he entered the employ of F. E. Devlin, Civil Engineer, of Wichita, Kans., and for the next two years he was Resident Engineer for the Company in charge of various municipal improvements in Caldwell, Kans., Waurika, Okla., and Englewood, Colo.

In 1925, when the Devlin Engineering Company was organized, with a branch office in Amarillo, Tex., Mr. Wells went with the writer to act as Engineer for the firm on projects in Western Texas and Eastern New Mexico. In this capacity he designed and supervised the construction of storm sewers and drainage works in Clarendon, Tex.; a sanitary sewer system in Brownfield, Tex.; water-works and sewer extensions and street paving in Portales, N. Mex.; street improvements at O'Donnel, Tex.; and street paving at Tulia, Tex. In 1927, Mr. Wells returned to Kansas to take charge of the installation of a sanitary sewer system in Cedarvale; he had completed this project a few months before his death at Clay Center, Kans.

While the foregoing account covers the work done by Mr. Wells during the short span of life allotted to him, mention should be made of some other things—the things of the spirit that remain and are even more permanent than the structures reared by this young engineer.

On completing his engineering training he chose to enter the ranks of those engineers who are fighting the battle of "Main Street"—fighting for the betterment of living conditions in the "small towns" of the Southwest. The writer recalls how, after many weary miles of driving over the plains of Texas, he would discuss with Mr. Wells the problems of these small towns that were being built. It was not the actual construction work in itself which was found so difficult, but the earlier task of the incorporation of these new towns with all the consequent handicaps that surround such an undertaking. The struggle with the apathy and ignorance of some of the inhabitants, the opposition in promoting mass meetings, and the discussions as to whether the bonds for the sewer and water systems—those elementary necessities of civilized living—should be voted for, were only some of the obstacles encountered.

In all these things, Mr. Wells showed not only great energy and ability, but also a rare understanding of these pioneer people from whom he sprang.

* Memoir prepared by Joseph Rady, Assoc. M. Am. Soc. C. E.

He loved the Southwest and its people, worked for them, and gave his life for them. He died of ptomaine poisoning, in the language of his native land, "with his boots on". He summed up his life's work aptly in closing his report to the City Commission of Clarendon, with these words: "It is our hope that this report may lead to a healthier and more beautiful Clarendon."

Mr. Wells was a member of the Delta Upsilon Fraternity and the Kansas Alumni Association.

His marriage, in June, 1925, to Norma McFadden, of Nevada, Mo., was the culmination of a romance of college days. He is survived by his widow and his parents.

Mr. Wells was elected a Junior of the American Society of Civil Engineers on March 12, 1923.

TRANSACTIONS

OF THE

AMERICAN SOCIETY OF CIVIL ENGINEERS

INDEX

VOLUME 93

1929

SUBJECT INDEX, PAGE 1980
AUTHOR INDEX, PAGE 1992

Titles of papers are in quotation marks when given with the author's name.

VOLUME 93

SUBJECT INDEX

ADDRESSES.

Address at the Annual Convention at Milwaukee, Wis., July 10, 1929. Anson Marston. 1736.

AGRICULTURE.

The effect of agricultural drainage upon flood run-off. 821.

ARCHES.

"Analysis of Arch Dams by the Trial Load Method." C. H. Howell and A. C. Jaquith. (With Discussion.) 1191.

"Load Distribution in High Arch Dams." R. A. Sutherland. (With Discussion.) 1623.

"Unusual Engineering Features of an Immense Theatre Building." R. McC. Beanfield. (With Discussion.) 152.

BAFFLE-PIERS.

See **SPILLWAY.**

BARS.

"The Great Yangtze Bar." A. V. Hugo von Heidenstam. (With Discussion.) 102.

BEAMS.

"Continuous — Over Three Spans." I. Oesterblom. 1367. Discussion: Frank S. Bailey, Clyde T. Morris, A. Lincoln Hyde, Harold E. Wessman, Robert M. Wilder, W. F. Way, H. M. Hadley, T. F. Hickerson, John I. Parcel, R. A. Caughey, John B. Snethlage, and T. D. Mylrea. 1384.

"Load Distribution in High Arch Dams." R. A. Sutherland. (With Discussion.) 1623.

BUILDINGS.

Bibliography on three-span construction in — . 1382.

"Continuous Beams Over Three Spans." I. Oesterblom. (With Discussion.) 1367.

"Unusual Engineering Features of an Immense Theatre Building." R. McC. Beanfield. 152. Discussion: Theodore Belzner, Fred A. Noetzli, A. Floris, and Laurence J. Waller. 182.

CANALS.

"The Head-Works of the Imperial Canal." C. E. Grunsky. (With Discussion.) 262.

CITIES.

The flood problem of New Orleans, La. 782.

COMMERCE.

The Port of Philadelphia. 240.

CONCRETE.

— used in O'Shaughnessy Dam. 1475, 1494, 1498.

"Methods Used in the Construction of Twelve Pre-Cast — Segments for the Alameda County, California, Estuary Subway." Alvin A. Horwege. 547. Discussion: Joseph A. Kitts. 565.

"Unusual Engineering Features of an Immense Theatre Building." R. McC. Beanfield. (With Discussion.) 152.

CONTINUOUS GIRDERS.

"Continuous Beams Over Three Spans." I. Oesterblom. (With Discussion.) 1367.

CONTRACTORS' PLANT.

Construction plant and methods for O'Shaughnessy Dam. 1463.

COSTS OF WORK.

— on O'Shaughnessy Dam and Reservoir. 1486.

DAMS.

"Analysis of Arch — by the Trial Load Method." C. H. Howell and A. C. Jaquith. 1191. Discussion: B. F. Jakobsen, George R. Rich, C. A. P. Turner, F. H. Constant, F. W. Hanna, L. J. Mensch, William Cain, Fred A. Noetzli, Fredrik Vogt, J. L. Savage and Ivan E. Houk, and A. Floris. 1226.

"Baffle-Pier Experiments on Models of Pit River —." I. C. Steele and R. A. Monroe. 451. Discussion: Edward C. Sherman, B. E. Torpen, Oren Reed, Harold K. Fox, Robert F. Ewald, F. W. Scheidenhelm, Ralph L. Parshall, H. W. Dennis, I. A. Winter, Paul Bauman, Morrough P. O'Brien, and J. C. Stevens. 481.

Bibliography on load distribution in high arch — . 1664.

"Hydraulic Studies and Operating Results on the Miami Flood Control System." C. H. Eiffert. (With Discussion.) 1583.

"Hydrostatic Uplift in Pervious Soils." H. de B. Parsons. 1317. Discussion: F. R. Harris, J. C. Meem, Eugene E. Halmos, T. R. Lawson, J. Vipond Davies, Lazarus White, T. Kennard Thomson, Daniel E. Moran, Alexander V. Gallogly, Allen Hazen, Charles Terzaghi, Frank S. Bailey, Charles R. Gow, E. L. Chandler, Martin J. McPike, Jacob Feld, J. Albert Holmes, and E. W. Lane. 1333.

"Load Distribution in High Arch — ." R. A. Sutherland. 1623. Discussion: William Cain, B. F. Jakobsen, Paul Bauman, A. Floris, Fredrik Vogt, and F. H. Constant. 1666.

Results of tests of core material from hydraulic-fill — . 352.

Study of the flow of water through soils with special reference to — . 314.

"The Conowingo Hydro-Electric Development on the Susquehanna River." Alexander Wilson, 3d. (With Discussion.) 979.

"The O'Shaughnessy Dam and Reservoir." John H. Gregory, C. B. Hoover, and C. B. Cornell. 1428. Discussion: J. Ernest Carman, B. F. Jakobsen, and W. H. Dittoe and G. Gale Dixon. 1493.

"Upward Pressures Under —: Experiments by the United States Bureau of Reclamation." Julian Hinds. 1527. Discussion: E. L. Chandler, B. F. Jakobsen, Charles Terzaghi, J. C. Stevens, F. W. Hanna, Ivan E. Houk, C. H. Howell, P. Wilhelm Werner, H. de B. Parsons, E. W. Lane, and C. E. Grunsky. 1551.

DRAINAGE.

The effect of agricultural — upon flood run-off. 821.

DREDGES.

See **DREDGING.**

DREDGING.

"Advances in Waterways Engineering During a Half Century." W. M. Black. (With Discussion.) 191.

"The Delaware River from Philadelphia to the Sea." F. C. Boggs. (With Discussion.) 218.

"The Great Yangtze Bar." A. V. Hugo von Heidenstam. (With Discussion.) 102.

DRY DOCKS.

Hydrostatic uplift pressure on Dry Dock No. 4 at Navy Yard, Brooklyn, N. Y. 1333, 1357.

EARTH PRESSURES.

"Hydrostatic Uplift in Pervious Soils." H. de B. Parsons. (With Discussion.) 1317.

"The Science of Foundations—Its Present and Future." Charles Terzaghi. (With Discussion.) 270.

EDUCATION.

"Engineering and Engineering — ": Address at the Annual Convention at Milwaukee, Wis., July 10, 1929. Anson Marston. 1736.

ELECTRIC LOCOMOTIVES.

— , Virginian Railway. 612.

ELECTRIC RAILWAYS.

"The Virginian Railway Electrification." George Gibbs. 571. Discussion: W. B. Potter, Marcel Japiot, L. K. Sillcox, I. Ofverholm, Ralph Budd, Guido Semenza, F. H. Shepard, E. R. Hill, Norman Litchfield, Sidney Withington, George J. Ray, John P. Hogan, and R. D. N. Simham. 628.

ELECTRIC TRANSMISSION.

See **POWER TRANSMISSION.**

ENGINEERING.

"— and — Education": Address at the Annual Convention at Milwaukee, Wis., July 10, 1929. Anson Marston. 1736.

FILTRATION.

"The Treatment of the Water Supply of the City of Columbus, Ohio." Charles P. Hoover. (With Discussion.) 1505.

FLOODS.

"Flood Control with Special Reference to the Mississippi River": A Symposium. Edgar Jadwin, C. McD. Townsend, H. C. Frankenfield, Nathan C. Grover, C. W. Kutz, E. F. McCarthy, William Kelly, Arthur E. Morgan, J. F. Coleman, Stuart C. Godfrey, Marcel Garsaud, C. E. Grunsky, Elwood Mead, F. W. Scheidenhelm, and Sherman M. Woodward and Floyd A. Nagler. (With Discussion.) 655.

"Hydraulic Studies and Operating Results on the Miami Flood Control System." C. H. Eiffert. 1583. Discussion: C. S. Bennett, Morris Knowles, Walter M. Smith, B. F. Jakobsen, K. L. De Blois, E. S. Lindley, C. S, Jarvis, E. W. Lane, and Charles H. Paul. 1602.

Levees as a means of flood control for the Mississippi River. 755.

"Maximum Flood Discharge in San Joaquin Valley, California." Oren Reed. 406. Discussion: Robert Follansbee, Harold K. Fox, and Charles H. Lee. 431.

Power as affecting flood control. 815.

The effect of agricultural drainage upon flood run-off. 821.

The relief outlets and by-passes of the Sacramento Valley flood-control project. 791.

FORESTS.

Forest cover as a factor in flood control. 716.

FOUNDATIONS.

Effect of freezing on — . 291.

Grouting the — of the O'Shaughnessy Dam. 1473.

"Hydrostatic Uplift in Pervious Soils." H. de B. Parsons. (With Discussion.) 1317.

"The Science of — : Its Present and Future." Charles Terzaghi. 270. Discussion: Charles R. Gow, E. P. Goodrich, Victor A. Endersby, George Paaswell, C. C. Williams, Herbert Chatley, J. Albert Holmes, Jacob Feld, W. S. Housel, Frank S. Bailey, T. L. Condron, Lazarus White, R. D. N. Simham, Frank A. Marston, Arthur M. Shaw, D. P. Krynine, S. P. Wing, M. L. Enger, and Albert Jorgensen. 302.

"Upward Pressures Under Dams: Experiments by the United States Bureau of Reclamation." Julian Hinds. (With Discussion.) 1527.

GROUTING.

— the foundation of the O'Shaughnessy Dam. 1473.

HARBORS.

"The Great Yangtze Bar." A. V. Hugo von Heidenstam. (With Discussion.) 102.
The Port of Philadelphia. 240.

HEAD-GATES.

"The Head-Works of the Imperial Canal." C. E. Grunsky. 262. Discussion: E. S. Lindley. 267.

HYDRAULIC JUMP.

— at Germantown Dam, Miami Conservancy District. 1589.

HYDRO-ELECTRIC POWER PLANTS.

See **POWER PLANTS.**

HYDROGRAPHY.

Fifty years of hydrographic surveying. 194.

IRRIGATION.

Reclamation as affecting flood control. 812.
"The Head-Works of the Imperial Canal." C. E. Grunsky. (With Discussion.) 262.

LEVEES.

"Flood Control with Special Reference to the Mississippi River": A Symposium. Edgar Jadwin, C. McD. Townsend, H. C. Frankenfield, Nathan C. Grover, C. W. Kutz, E. F. McCarthy, William Kelly, Arthur E. Morgan, J. F. Coleman, Stuart C. Godfrey, Marcel Garsaud, C. E. Grunsky, Elwood Mead, F. W. Scheidenhelm, and Sherman M. Woodward and Floyd A. Nagler. (With Discussion.) 655.
— as a means of flood control for the Mississippi River. 755.
The work of the Mississippi River Commission. 679.

MEMOIRS OF DECEASED MEMBERS.

Abbot, Frederic Vaughan. 1745.
Ackhart, Andrew Lewis. 1923.
Adams, Frederick Baird. 1925.
Adams, Henry Sewall. 1747.
Anderson, Robert Paul. 1750.
Angier, Walter Eugene. 1752.
Balch, Leland Rella. 1754.
Baldwin, Hiram Ellsworth. 1926.
Ball, John Wesley. 1756.
Barr, John Toner. 1758.
Baxter, George Strong. 1760.
Bayly, Howard Paul. 1928.
Beahan, Willard. 1762.
Bissell, Hezekiah. 1764.
Bixby, William Herbert. 1768.

Blunt, William Titcomb. 1772.
Boynton, George Herbert. 1774.
Bremner, George Hampton. 1777.
Burke, William Charles. 1779.
Burnell, Eugene Dickinson. 1781.
Burt, Henry Jackson. 1784.
Carll, David Sylvanus. 1786.
Churchill, John Charles. 1788.
Clifford, Walter Woodbridge. 1790.
Coughlin, William Godley. 1792.
Crane, Clarence Austin. 1794.
Crawford, Merlin Cross. 1929.
Cummings, Elmore David. 1796.
Curry, Charles Kenneth. 1930.
Davis, Robert Waite. 1931.
Dawley, William Sanborn. 1798.
Donnelly, Joseph Francis Sinnott. 1933.
Dousman, James Hopkins. 1801.
Edwards, Guy Banker. 1935.
Edwards, John Martynn Grenfell. 1937.
Elliott, Howard. 1965.
Ely, John Stanton. 1803.
Felt, Charles Frederick Wilson. 1805.
Flagg, Josiah Foster. 1808.
Fletcher, Austin Bradstreet. 1810.
Giesting, Frank Alexander. 1939.
Goethals, George Washington. 1813.
Going, Alvah Seymour. 1818.
Haight, Silas Maxwell. 1820.
Harrison, William Burr. 1822.
Helland, Hans. 1824.
Hendricks, Kearney Everett. 1825.
Heyworth, James Ormerod. 1826.
Holbrook, John Byers. 1828.
Horne, Harold Wellington. 1829.
Howell, David Janney. 1831.
Howells, Julius Merriam. 1834.
Hubbard, Frank James. 1837.
Humphrey, Richard Lewis. 1839.
Jones, Fred Atwood. 1843.
Joslin, Harold Vincent. 1845.
Kelley, Howard George. 1847.
Kennedy, James Henry. 1854.
Kent, Willard. 1858.
Kircher, Paul. 1859.
Klyce, Battle Hargrove. 1861.
Lackey, Oscar Francis. 1863.
Larson, Clarence Melrose. 1941.
Leavitt, Charles Wellford. 1865.
Leavitt, Frank McDowell. 1867.
Leckliter, Walter Harlan. 1943.
Lehlbach, Gustav. 1871.

Levy, Alfred. 1945.
Low, Emile. 1874.
Maxson, Frank Oscar. 1876.
Midolo, Mario John. 1975.
Nagle, James C. 1878.
Nauman, George. 1880.
Noska, George Albert. 1881.
Onstott, Virgil Samms. 1947.
Palmer, William Pendleton. 1969.
Pearson, Edward Jones. 1882.
Porter, James Madison, 3d. 1948.
Price, William Gunn. 1885.
Proal, Arthur Breese, Jr. 1887.
Quincy, Charles Frederick. 1971.
Reeves, Harley Edson. 1889.
Riggs, Morris John. 1890.
Rose, John Gansovertte. 1950.
Rose, Rudolf Viedt. 1892.
Ryan, Michael Healey. 1953.
Sands, Edward Emmet. 1893.
Schoenberger, George Christian. 1955.
Scofield, Glenn Mason. 1895.
Scott, Addison Moffat. 1897.
Spencer, Walter Tuttle. 1958.
Staats, John Henry. 1899.
Staley, Cady. 1900.
Stewart, John Truesdale. 1902.
Stewart, Lucern S. 1972.
Strickland, Kirby Lee. 1904.
Tallman, Paul Bertram. 1906.
Tays, Eugene Augustus Hoffman. 1908.
Thorndike, Sturgis Hooper. 1910.
Tillman, Frank Andrew. 1960.
Tully, Edward John. 1962.
Van Petten, Albert Alexander. 1963.
Warner, Edwin Hall. 1913.
Weller, Francis Repetti. 1916.
Wells, William Bacon. 1977.
Wheeler, Bertrand Thorp. 1919.
Wingfield, Nisbet. 1921.

MODELS.

"Baffle-Pier Experiments on — of Pit River Dams." I. C. Steele and R. A. Monroe. (With Discussion.) 451.

MOMENTS.

"Continuous Beams Over Three Spans." I. Oesterblom. (With Discussion.) 1367.

NAVIGATION.

Improvement of — in relation to flood control. 762.

PILE-DRIVING.

"The Science of Foundations—Its Present and Future." Charles Terzaghi. (With Discussion.) 270.

PILES.

"The Science of Foundations—Its Present and Future." Charles Terzaghi. (With Discussion.) 270.

PORTS.

See **HARBORS.**

POWER HOUSES.

See **POWER PLANTS.**

POWER PLANTS.

Narrows Power Plant of the Virginian Railway. 584.

"Re-Designing Catawba Station for Service on a Large Transmission System." W. S. Lee. 127. Discussion: H. A. Hageman, H. V. Joslin, and T. H. Hogg. 145.

"The Conowingo Hydro-Electric Development on the Susquehanna River." Alexander Wilson, 3d. (With Discussion.) 979.

POWER STATIONS.

See **POWER PLANTS.**

POWER TRANSMISSION.

High-tension transmission, Virginian Railway. 596.

PUMPS.

"New Theory for the Centrifugal Pump." Allen F. Sherzer. 1. Discussion: J. W. MacMeeken, Thomson Kingsford, Sanford A. Moss, P. L. Reed, Max M. Munk, Elmo G. Harris, O. Gruenberger, Elmer G. Hooper, F. Theodore Mavis, Mario Medici, A. H. Blaisdell, Morrough P. O'Brien, R. T. Liddicoat, Lloyd H. Donnell, S. Timoshenko, and A. B. Cox. 30.

RAILROAD ELECTRIFICATION.

See **ELECTRIC RAILWAYS.**

RAILROADS.

Relocation of Pennsylvania tracks adjacent to Conowingo Power Development. 981.

"The Virginian Railway Electrification." George Gibbs. (With Discussion.) 571.

RAINFALL.

"Maximum Flood Discharge in San Joaquin Valley, California." Oren Reed. (With Discussion.) 406.

— characteristics of the Mississippi Drainage Basin. 675.

— of the Miami Conservancy District. 1597.

"The Conowingo Hydro-Electric Development on the Susquehanna River." Alexander Wilson, 3d. (With Discussion.) 979.

REINFORCED CONCRETE.

"Continuous Beams Over Three Spans." I. Oesterblom. (With Discussion.) 1367.

RESERVOIRS.

Power as affecting flood control. 815.

— for Mississippi Valley flood protection. 709, 724.

"Silting of the Lake at Austin, Texas." T. U. Taylor. (With Discussion.) 1681.

The basis of the case against — for Mississippi flood control. 737.

"The O'Shaughnessy Dam and Reservoir." John H. Gregory, C. B. Hoover, and C. B. Cornell. (With Discussion.) 1428.

The work of the Mississippi River Commission. 679.

RIVER REGULATION.

See **RIVERS.**

RIVERS.

"Advances in Waterways Engineering During a Half Century." W. M. Black. (With Discussion.) 191.

"Flood Control with Special Reference to the Mississippi River": A. Symposium. Edgar Jadwin, C. McD. Townsend, H. C. Frankenfield, Nathan C. Grover, C. W. Kutz, E. F. McCarthy, William Kelly, Arthur E. Morgan, J. F. Coleman, Stuart C. Godfrey, Marcel Garsaud, C. E. Grunsky, Elwood Mead, F. W. Scheidenhelm, and Sherman M. Woodward and Floyd A. Nagler. 655. Discussion: J. P. Snow, Frank B. Maltby, Morris Knowles, Charles H. Paul, F. G. Jonah, C. S. Jarvis, Edward H. Schulz, Q. C. Ayres, H. S. Gladfelter, Byron E. White, W. G. Price, Oren Reed, Harold C. Fiske, Charles F. Brooks, William Gerig, C. H. Eiffert, Sherman M. Woodward, W. J. Schlick, S. L. Moyer, William M. Hall, C. E. Ramser, T. Kennard Thomson, W. E. Elam, R. W. Davenport, and E. W. Lane. 840.

Levees as a means of flood control for the Mississippi River. 755.

"Maximum Flood Discharge in San Joaquin Valley, California." Oren Reed. (With Discussion.) 406.

"The Delaware River from Philadelphia to the Sea." F. C. Boggs. 218. Discussion: Charles M. Spofford, Henry J. Sherman, Earl I. Brown, Harry Taylor, F. C. Warner, and William Gerig. 246.

"The Great Yangtze Bar." A. V. Hugo von Heidenstam. 102. Discussion: A. W. Robinson, and R. L. Vaughn. 120.

The relief outlets and by-passes of the Sacramento Valley flood-control project. 791.

The work of the Mississippi River Commission. 679.

RUN-OFF.

"Flood Control with Special Reference to the Mississippi River": A Symposium. Edgar Jadwin, C. McD. Townsend, H. C. Frankenfield, Nathan C. Grover, C. W. Kutz, E. F. McCarthy, William Kelly, Arthur E. Morgan,

J. F. Coleman, Stuart C. Godfrey, Marcel Garsaud, C. E. Grunsky, Elwood Mead, F. W. Scheidenhelm, and Sherman M. Woodward and Floyd A. Nagler. (With Discussion.) 655.
"Maximum Flood Discharge in San Joaquin Valley, California." Oren Reed. (With Discussion.) 400.
— characteristics of the Mississippi River. 690.
— of the Miami Conservancy District. 1597.
"The Conowingo Hydro-Electric Development on the Susquehanna River." Alexander Wilson, 3d. (With Discussion.) 979.
The effect of agricultural drainage upon flood — . 821.
Water-shed and flow of the Scioto River. 1431.

SEEPAGE.

"Upward Pressures Under Dams: Experiments by the United States Bureau of Reclamation." Julian Hinds. (With Discussion.) 1527.

SILT.

— deposited on the Great Yangtze Bar. 118.
"Silting of the Lake at Austin, Texas." T. U. Taylor. 1681. Discussion: L. M. Lawson, C. H. Eiffert, Oren Reed, Charles Schultz, John B. Hawley, H. F. Robinson, Julian Montgomery, Kirk Bryan, J. C. Stevens, P. A. Welty, Banks McLaurin, R. G. Tyler, E. C. H. Bantel, R. F. Walter, William Jenner Powell, and R. I. Meeker. 1690.
"The Head-Works of the Imperial Canal." C. E. Grunsky. (With Discussion.) 262.

SOILS.

"Hydrostatic Uplift in Pervious — ." H. de B. Parsons. (With Discussion.) 1317.
"The Science of Foundations—Its Present and Future." Charles Terzaghi. (With Discussion.) 270.

SPILLWAY.

"Baffle-Pier Experiments on Models of Pit River Dams." I. C. Steele and R. A. Monroe. (With Discussion.) 451.
Design of — of O'Shaughnessy Dam. 1451.
Relief spillways at New Orleans. 785.
— for new Catawba Station dam. 132.
The relief outlets and by-passes of the Sacramento Valley flood-control project. 791.

STEEL.

"Unusual Engineering Features of an Immense Theatre Building." R. McC. Beanfield. (With Discussion.) 152.

STREAM FLOW.

See **WATER, FLOW OF, IN OPEN CHANNELS.**

STRESS.

Bibliography on three-span construction in buildings. 1382.

"Continuous Beams Over Three Spans." I. Oesterblom. (With Discussion.) 1367.

"Load Distribution in High Arch Dams." R. A. Sutherland. (With Discussion.) 1623.

"Unusual Engineering Features of an Immense Theatre Building." R. McC. Beanfield. (With Discussion.) 152.

SUBWAYS.

"Methods Used in the Construction of Twelve Pre-Cast Concrete Segments for the Alameda County, California, Estuary Subway." Alvin A. Horwege. (With Discussion.) 547.

SURVEYING.

Fifty years of hydrographic — . 194.

THEATRES.

"Unusual Engineering Features of an Immense Theatre Building." R. McC. Beanfield. (With Discussion.) 152.

TIDES.

— and currents in the Delaware River. 220.

TRUSSES.

"Unusual Engineering Features of an Immense Theatre Building." R. McC. Beanfield. (With Discussion.) 152.

TUNNELS.

"Methods Used in the Construction of Twelve Pre-Cast Concrete Segments for the Alameda County, California, Estuary Subway." Alvin A. Horwege. (With Discussion.) 547.

WATER.

"The Treatment of the — Supply of the City of Columbus, Ohio." Charles P. Hoover. 1505. Discussion: F. H. Waring, George W. Fuller, Robert Spurr Weston, and C. W. Foulk. 1519.

WATER, FLOW OF, IN OPEN CHANNELS.

"Hydraulic Studies and Operating Results on the Miami Flood Control System." C. H. Eiffert. (With Discussion.) 1583.

"Maximum Flood Discharge in San Joaquin Valley, California." Oren Reed. (With Discussion.) 406.

"The Delaware River from Philadelphia to the Sea." F. C. Boggs. (With Discussion.) 218.

WATER, FLOW OF, IN PIPES.

"Hydraulic Studies and Operating Results on the Miami Flood Control System." C. H. Eiffert. (With Discussion.) 1583.

WATER, FLOW OF, OVER DAMS AND WEIRS.

"Baffle-Pier Experiments on Models of Pit River Dams." I. C. Steele and R. A. Monroe. (With Discussion.) 451.

"Precise Weir Measurements." Ernest W. Schoder and Kenneth B. Turner. 999. Discussion: H. W. King, Floyd A. Nagler, A. Streiff, R. L. Parshall, W. S. Pardoe, R. E. Ballester, Gardner S. Williams, Th. Rehbock, Erik G. W. Lindquist, and Clemens Herschel. 1111.

WATER POWER.

Power as affecting flood control. 815.

"The Conowingo Hydro-Electric Development on the Susquehanna River." Alexander Wilson, 3d. 970. Discussion: William C. L. Eglin, J. V. Hogan, A. W. Clark, and H. A. Hageman. 979.

WATER PRESSURES.

"Hydrostatic Uplift in Pervious Soils." H. de B. Parsons. (With Discussion.) 1317.

"Upward Pressures Under Dams: Experiments by the United States Bureau of Reclamation." Julian Hinds. (With Discussion.) 1527.

WATER PURIFICATION.

See **WATER.**

WATER SOFTENING.

See **WATER.**

WATER STORAGE.

"The O'Shaughnessy Dam and Reservoir." John H. Gregory, C. B. Hoover, and C. B. Cornell. (With Discussion.) 1428.

WATERWAYS.

"Advances in — Engineering During a Half Century." W. M. Black. 191. Discussion: C. L. Hall, and DeWitt D. Barlow. 214.

WEIRS.

See **WATER, FLOW OF, OVER DAMS AND WEIRS.**

AUTHOR INDEX

ABBOT, FREDERIC VAUGHAN.
Memoir of. 1745.

ACKHART, ANDREW LEWIS.
Memoir of. 1923.

ADAMS, FREDERICK BAIRD.
Memoir of. 1925.

ADAMS, HENRY SEWALL.
Memoir of. 1747.

ANDERSON, ROBERT PAUL.
Memoir of. 1750.

ANGIER, WALTER EUGENE.
Memoir of. 1752.

AYRES, Q. C.
Mississippi River flood control. 878.

BAILEY, FRANK S.
Continuous beams over three spans. 1384.
Hydrostatic uplift in pervious soils. 1353.
The science of foundations. 330.

BALCH, LELAND RELLA.
Memoir of. 1754.

BALDWIN, HIRAM ELLSWORTH.
Memoir of. 1926.

BALL, JOHN WESLEY.
Memoir of. 1756.

BALLESTER, R. E.
Precise weir measurements. 1136.

BANTEL, E. C. H.
Silting of lake at Austin, Texas. 1717.

BARLOW, DeWITT D.
Progress in waterways engineering. 214.

BARR, JOHN TONER.
Memoir of. 1758.

BAUMAN, PAUL.
Experiments on models of Pit River dams. 518.
Load distribution in high arch dams. 1670.

BAYLY, HOWARD PAUL.
Memoir of. 1928.

BAXTER, GEORGE STRONG.
Memoir of. 1760.

BEAHAN, WILLARD.
Memoir of. 1762.

BEANFIELD, R. McC.
"Unusual Engineering Features of an Immense Theatre Building." 152.

BELZNER, THEODORE.
Engineering features of a theatre building. 182.

BENNETT, C. S.
Results of Miami flood control system. 1602.

BISSELL, HEZEKIAH.
Memoir of. 1764.

BIXBY, WILLIAM HERBERT.
Memoir of. 1768.

BLACK, W. M.
"Advances in Waterways Engineering During a Half Century." 191.

BLAISDELL, A. H.
New theory for the centrifugal pump. 63.

BLUNT, WILLIAM TITCOMB.
Memoir of. 1772.

BOGGS, F. C.
"The Delaware River from Philadelphia to the Sea." 218.

BOYNTON, GEORGE HERBERT.
Memoir of. 1774.

BREMNER, GEORGE HAMPTON.
Memoir of. 1777.

BROOKS, CHARLES F.
Mississippi River flood control. 894.

BROWN, EARL I.
Delaware River improvement. 249.

BRYAN, KIRK.
Silting of lake at Austin, Texas. 1703.

BUDD, RALPH.
The Virginian Railway electrification. 634.

BURKE, WILLIAM CHARLES.
Memoir of. 1779.

BURNELL, EUGENE DICKINSON.
Memoir of. 1781.

BURT, HENRY JACKSON.
Memoir of. 1784.

CAIN, WILLIAM.
Arch dam analysis by trial load method. 1265.
Load distribution in high arch dams. 1666.

CARLL, DAVID SYLVANUS.
Memoir of. 1786.

CARMAN, J. ERNEST.
The O'Shaughnessy Dam and Reservoir. 1493.

CAUGHEY, R. A.
Continuous beams over three spans. 1410.

CHANDLER, E. L.
Hydrostatic uplift in pervious soils. 1356.
Upward pressures under dams. 1551.

CHATLEY, HERBERT.
The science of foundations. 313.

CHURCHILL, JOHN CHARLES.
Memoir of. 1788.

CLARK, A. W.
Conowingo hydro-electric development. 990.

CLIFFORD, WALTER WOODBRIDGE.
Memoir of. 1790.

COLEMAN, J. F.
"Levees as a Means of Flood Control for the Mississippi River." 755.

CONDRON, T. L.
The science of foundations. 331.

CONSTANT, F. H.
Arch dam analysis by trial load method. 1241.
Load distribution in high arch dams. 1677.

CORNELL, C. B.
"The O'Shaughnessy Dam and Reservoir." 1428.

COUGHLIN, WILLIAM GODLEY.
Memoir of. 1792.

COX, A. B.
New theory for the centrifugal pump. 76.

CRANE, CLARENCE AUSTIN.
Memoir of. 1794.

CRAWFORD, MERLIN CROSS.
Memoir of. 1929.

CUMMINGS, ELMORE DAVID.
Memoir of. 1796.

CURRY, CHARLES KENNETH.
Memoir of. 1930.

DAVENPORT, R. W.
Mississippi River flood control. 947.

DAVIES, J. VIPOND.
Hydrostatic uplift in pervious soils. 1341.

DAVIS, ROBERT WAITE.
Memoir of. 1931.

DAWLEY, WILLIAM SANBORN.
Memoir of. 1798.

De BLOIS, K. L.
Results of Miami flood control system. 1610.

DENNIS, H. W.
Experiments on models of Pit River dams. 511.

DITTOE, W. H.
The O'Shaughnessy Dam and Reservoir. 1495.

DIXON, G. GALE.
The O'Shaughnessy Dam and Reservoir. 1495.

DONNELL, LLOYD H.
New theory for the centrifugal pump. 71.

DONNELLY, JOSEPH FRANCIS SINNOTT.
Memoir of. 1933.

DOUSMAN, JAMES HOPKINS.
Memoir of. 1801.

EDWARDS, GUY BANKER.
Memoir of. 1935.

EDWARDS, JOHN MARTYNN GRENFELL.
Memoir of. 1937.

EGLIN, WILLIAM C. L.
Conowingo hydro-electric development. 979.

EIFFERT, C. H.
"Hydraulic Studies and Operating Results on the Miami Flood Control System." 1583.
Mississippi River flood control. 900.
Silting of lake at Austin, Texas. 1690.

ELAM, W. E.
Mississippi River flood control. 937.

ELLIOTT, HOWARD.
Memoir of. 1965.

ELY, JOHN STANTON.
Memoir of. 1803.

ENDERSBY, VICTOR A.
The science of foundations. 304.

ENGER, M. L.
The science of foundations. 371.

EWALD, ROBERT F.
Experiments on models of Pit River dams. 499.

FELD, JACOB.
Hydrostatic uplift in pervious soils. 1360.
The science of foundations. 319.

FELT, CHARLES FREDERICK WILSON.
Memoir of. 1805.

FISKE, HAROLD C.
Mississippi River flood control. 891.

FLAGG, JOSIAH FOSTER.
Memoir of. 1808.

FLETCHER, AUSTIN BRADSTREET.
Memoir of. 1810.

FLORIS, A.
Arch dam analysis by trial load method. 1296.
Engineering features of a theatre building. 183.
Load distribution in high arch dams. 1675.

FOLLANSBEE, ROBERT.
Flood discharge in San Joaquin Valley, Calif. 431.

FOULK, C. W.
Water supply purification, Columbus, Ohio. 1525.

FOX, HAROLD K.
Experiments on models of Pit River dams. 498.
Flood discharge in San Joaquin Valley, Calif. 431.

FRANKENFIELD, H. C.
"Rainfall Characteristics of the Mississippi Drainage Basin." 675.

FULLER, GEORGE W.
Water supply purification, Columbus, Ohio. 1522.

GALLOGLY, ALEXANDER V.
Hydrostatic uplift in pervious soils. 1344.

GARSAUD, MARCEL.
"The Flood Problem of New Orleans, Louisiana." 782.

GERIG, WILLIAM.
Delaware River improvement. 258.
Mississippi River flood control. 898.

GIBBS, GEORGE.
"The Virginian Railway Electrification." 571.

GIESTING, FRANK ALEXANDER.
Memoir of. 1939.

GLADFELTER, H. S.
Mississippi River flood control. 879.

GODFREY, STUART C.
"Improvement of Navigation in Relation to Flood Control." 762.

GOETHALS, GEORGE WASHINGTON.
Memoir of. 1813.

GOING, ALVAH SEYMOUR.
Memoir of. 1818.

GOODRICH, E. P.
The science of foundations. 303.

GOW, CHARLES R.
Hydrostatic uplift in pervious soils. 1353.
The science of foundations. 302.

GREGORY, JOHN H.
"The O'Shaughnessy Dam and Reservoir." 1428.

GROVER, NATHAN C.
"Run-Off Characteristics of the Mississippi River Drainage Basin." 690.

GRUENBERGER, O.
New theory for the centrifugal pump. 43.

GRUNSKY, C. E.
"The Head-Works of the Imperial Canal." 262.
"The Relief Outlets and By-Passes of the Sacramento Valley Flood-Control Project." 791.
Upward pressures under dams. 1578.

HADLEY, H. M.
Continuous beams over three spans. 1396.

HAGEMAN, H. A.
Conowingo hydro-electric development. 993.
Re-design of Catawba Station. 145.

HAIGHT, SILAS MAXWELL.
Memoir of. 1820.

HALL, C. L.
Progress in waterways engineering. 214.

HALL, WILLIAM M.
Mississippi River flood control. 919.

HALMOS, EUGENE E.
Hydrostatic uplift in pervious soils. 1339.

HANNA, F. W.
Arch dam analysis by trial load method. 1246.
Upward pressures under dams. 1564.

HARRIS, ELMO G.
New theory for the centrifugal pump. 37.

HARRIS, F. R.
Hydrostatic uplift in pervious soils. 1333.

HARRISON, WILLIAM BURR.
Memoir of. 1822.

HAWLEY, JOHN B.
Silting of lake at Austin, Texas. 1693.

HAZEN, ALLEN.
Hydrostatic uplift in pervious soils. 1345.

HELLAND, HANS.
Memoir of. 1824.

HENDRICKS, KEARNEY EVERETT.
Memoir of. 1825.

HERSCHEL, CLEMENS.
Precise weir measurements. 1176.

HEYWORTH, JAMES ORMEROD.
Memoir of. 1826.

HICKERSON, T. F.
Continuous beams over three spans. 1405.

HILL, E. R.
The Virginian Railway electrification. 636.

HINDS, JULIAN.
"Upward Pressures Under Dams: Experiments by the United States Bureau of Reclamation." 1527.

HOGAN, J. V.
Conowingo hydro-electric development. 981.

HOGAN, JOHN P.
The Virginian Railway electrification. 642.

HOGG, T. H.
Re-design of Catawba Station. 149.

HOLBROOK, JOHN BYERS.
Memoir of. 1828.

HOLMES, J. ALBERT.
Hydrostatic uplift in pervious soils. 1362.
The science of foundations. 314.

HOOPER, ELMER G.
New theory for the centrifugal pump. 44.

HOOVER, C. B.
"The O'Shaughnessy Dam and Reservoir." 1428.

HOOVER, CHARLES P.
"The Treatment of the Water Supply of the City of Columbus, Ohio." 1505.

HORNE, HAROLD WELLINGTON.
Memoir of. 1829.

HORWEGE, ALVIN A.
"Methods Used in the Construction of Twelve Pre-Cast Concrete Segments for the Alameda County, California, Estuary Subway." 547.

HOUK, IVAN E.
Arch dam analysis by trial load method. 1289.
Upward pressures under dams. 1565.

HOUSEL, W. S.
The science of foundations. 322.

HOWELL, C. H.
"Analysis of Arch Dams by the Trial Load Method." 1191.
Upward pressures under dams. 1571.

HOWELL, DAVID JANNEY.
Memoir of. 1831.

HOWELLS, JULIUS MERRIAM.
Memoir of. 1834.

HUBBARD, FRANK JAMES.
Memoir of. 1837.

HUMPHREY, RICHARD LEWIS.
Memoir of. 1839.

HYDE, A. LINCOLN.
Continuous beams over three spans. 1386.

JADWIN, EDGAR.
"Flood Control with Special Reference to the Mississippi River: Introduction." 657.

JAKOBSEN, B. F.
Arch dam analysis by trial load method. 1226.
Load distribution in high arch dams. 1668.
Results of Miami flood control system. 1607.
The O'Shaughnessy Dam and Reservoir. 1494.
Upward pressures under dams. 1558.

JAPIOT, MARCEL.
The Virginian Railway electrification. 629.

JAQUITH, A. C.
"Analysis of Arch Dams by the Trial Load Method." 1191.

JARVIS, C. S.
Mississippi River flood control. 858.
Results of Miami flood control system. 1614.

JONAH, F. G.
Mississippi River flood control. 854.

JONES, FRED ATWOOD.
Memoir of. 1843.

JORGENSEN, ALBERT.
The science of foundations. 378.

JOSLIN, HAROLD VINCENT.
Memoir of. 1845.
Re-design of Catawba Station. 148.

KELLEY, HOWARD GEORGE.
Memoir of. 1847.

KELLY, WILLIAM.
"Reservoirs for Mississippi Valley Flood Protection." 724

KENNEDY, JAMES HENRY.
Memoir of. 1854.

KENT, WILLARD.
Memoir of. 1858.

KING, H. W.
Precise weir measurements. 1111.

KINGSFORD, THOMSON.
New theory for the centrifugal pump. 33.

KIRCHER, PAUL.
Memoir of. 1859.

KITTS, JOSEPH A.
Methods of pre-casting concrete segments for Alameda County, California, Estuary Subway. 565.

KLYCE, BATTLE HARGROVE.
Memoir of. 1859.

KNOWLES, MORRIS.
Mississippi River flood control. 845.
Results of Miami flood control system. 1605.

KRYNINE, D. P.
The science of foundations. 357.

KUTZ, C. W.
"The Work of the Mississippi River Commission." 697.

LACKEY, OSCAR FRANCIS.
Memoir of. 1863.

LANE, E. W.
Hydrostatic uplift in pervious soils. 1363.
Mississippi River flood control. 949.
Results of Miami flood control system. 1615.
Upward pressures under dams. 1576.

LARSON, CLARENCE MELROSE.
Memoir of. 1941.

LAWSON, L. M.
Silting of lake at Austin, Texas. 1690.

LAWSON, T. R.
Hydrostatic uplift in pervious soils. 1340.

LEAVITT, CHARLES WELLFORD.
Memoir of. 1865.

LEAVITT, FRANK McDOWELL.
Memoir of. 1867.

LECKLITER, WALTER HARLAN.
Memoir of. 1943.

LEE, CHARLES H.
Flood discharge in San Joaquin Valley, Calif. 433.

LEE, W. S.
"Re-Designing Catawba Station for Service on a Large Transmission System." 127.

LEHLBACH, GUSTAV.
Memoir of. 1871.

LEVY, ALFRED.
Memoir of. 1945.

LIDDICOAT, R. T.
New theory for the centrifugal pump. 67.

LINDLEY, E. S.
Head-works of the Imperial Canal. 267.
Results of Miami flood control system. 1612.

LINDQUIST, ERIK G. W.
Precise weir measurements. 1163.

LITCHFIELD, NORMAN.
The Virginian Railway electrification. 638.

LOW, EMILE.
Memoir of. 1874.

MacMEEKEN, J. W.
New theory for the centrifugal pump. 30.

McCARTHY, E. F.
"Forest Cover as a Factor in Flood Control." 716.

McLAURIN, BANKS.
Silting of lake at Austin, Texas. 1713.

McPIKE, MARTIN J.
Hydrostatic uplift in pervious soils. 1357.

MALTBY, FRANK B.
Mississippi River flood control. 841.

MARSTON, ANSON.
"Engineering and Engineering Education": Address at the Annual Convention at Milwaukee, Wis., July 10, 1929. 1736.

MARSTON, FRANK A.
The science of foundations. 350.

MAVIS, F. THEODORE.
New theory for the centrifugal pump. 46.

MAXSON, FRANK OSCAR.
Memoir of. 1876.

MEAD, ELWOOD.
"Reclamation as Affecting Flood Control." 812.

MEDICI, MARIO.
New theory for the centrifugal pump. 51.

MEEKER, R. I.
Silting of lake at Austin, Texas. 1731.

MEEM, J. C.
Hydrostatic uplift in pervious soils. 1334.

MENSCH, L. J.
Arch dam analysis by trial load method. 1250.

MIDOLO, MARIO JOHN.
Memoir of. 1975.

MONROE, R. A.
"Baffle-Pier Experiments on Models of Pit River Dams." 451.

MONTGOMERY, JULIAN.
Silting of lake at Austin, Texas. 1703.

MORAN, DANIEL E.
Hydrostatic uplift in pervious soils. 1344.

MORGAN, ARTHUR E.
"The Basis of the Case Against Reservoirs for Mississippi Flood Control." 737.

MORRIS, CLYDE T.
Continuous beams over three spans. 1385.

MOSS, SANFORD A.
New theory for the centrifugal pump. 34.

MOYER, S. L.
Mississippi River flood control. 907.

MUNK, MAX M.
New theory for the centrifugal pump. 37.

MYLREA, T. D.
Continuous beams over three spans. 1421.

NAGLE, JAMES C.
Memoir of. 1878.

NAGLER, FLOYD A.
Precise weir measurements. 1114.
"The Effect of Agricultural Drainage upon Flood Run-Off." 821.

NAUMAN, GEORGE.
Memoir of. 1880.

NOETZLI, FRED A.
Arch dam analysis by trial load method. 1270.
Engineering features of a theatre building. 182.

NOSKA, GEORGE ALBERT.
Memoir of. 1881.

O'BRIEN, MORROUGH P.
Experiments on models of Pit River dams. 528.
New theory for the centrifugal pump. 66.

OESTERBLOM, I.
"Continuous Beams Over Three Spans." 1367.

OFVERHOLM, I.
The Virginian Railway electrification. 633.

ONSTOTT, VIRGIL SAMMS.
Memoir of. 1947.

PAASWELL, GEORGE.
The science of foundations. 305.

PALMER, WILLIAM PENDLETON.
Memoir of. 1969.

PARCEL, JOHN I.
Continuous beams over three spans. 1410.

PARDOE, W. S.
Precise weir measurements. 1130.

PARSHALL, RALPH L.
Experiments on models of Pit River dams. 507.
Precise weir measurements. 1127.

PARSONS, H. de B.
"Hydrostatic Uplift in Pervious Soils." 1317.
Upward pressures under dams. 1574.

PAUL, CHARLES H.
Mississippi River flood control. 853.
Results of Miami flood control system. 1616.

PEARSON, EDWARD JONES.
Memoir of. 1882.

PORTER, JAMES MADISON, 3d,
Memoir of. 1948.

POTTER, W. B.
The Virginian Railway electrification. 628.

POWELL, WILLIAM JENNER.
Silting of lake at Austin, Texas. 1728.

PRICE, WILLIAM GUNN.
Memoir of. 1885.
Mississippi River flood control. 887.

PROAL, ARTHUR BREESE, Jr.
Memoir of. 1887.

QUINCY, CHARLES FREDERICK.
Memoir of. 1971.

RAMSER, C. E.
Mississippi River flood control. 933.

RAY, GEORGE J.
The Virginian Railway electrification. 641.

REED, OREN.
Experiments on models of Pit River dams. 497.
"Maximum Flood Discharge in San Joaquin Valley, California." 406.
Mississippi River flood control. 890.
Silting of lake at Austin, Texas. 1691.

REED, P. L.
New theory for the centrifugal pump. 35.

REEVES, HARLEY EDSON.
Memoir of. 1889.

REHBOCK, TH.
Precise weir measurements. 1143.

RICH, GEORGE R.
Arch dam analysis by trial load method. 1229.

RIGGS, MORRIS JOHN.
Memoir of. 1890.

ROBINSON, A. W.
The great Yangtze Bar. 120.

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